IRC:SP:37-2010

GUIDELINES FOR EVALUATION OF LOAD CARRYING CAPACITY OF BRIDGES

(*First Revision*)

INDIAN ROADS CONGRESS

2010

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GUIDELINES FOR EVALUATION OF LOAD CARRYING CAPACITY OF BRIDGES

1 INTRODUCTION

1.1 Revision of IRC:SP:37 "Guidelines for Evaluation of Load Carrying Capacity of Bridges" has been under the consideration of Maintenance and Rehabilitation Committee, then named as B-9 Committee, since December, 2004.

1.2 In the recent past, several mother Codes and Supplementary Guidelines have either been revised, modified or are under revision.

The Motor Vehicles Act and Regulations 1988, revised the limit of axle loads, GVW and new dimensional limits for actual vehicles. Also with the increasing cost of fuel, POL, the tendency of transport operators has been to carry increased freight resulting in the overloading. This has been observed and measured in a number of surveys conducted on NHs, and SHs.

IRC:6 "Standard Specifications and Code of Practice for Road Bridges, Section - II" has also been extensively revised.

In view of the changes and modifications in the associated Codes and Motor Vehicles Act, it has became necessary to revise IRC:SP:37.

1.3 The Maintenance and Rehabilitation Committee, now named as B-8 Committee, was reconstituted in 2009 with the following personnel:

Narain, A.D.	...	Convenor
Manjure, P.Y.	...	Co-Convenor
Thandavan, K.B.	...	Member Secretary

Members

Bagish, Dr. B.P.	Koshi, Ninan
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Secretary General, IRC	(Indoria, R.P.)

1.4 The erstwhile B-9 Committee could discuss the revision and finalized only a few clauses. The newly constituted B-8 Committee thereafter took up the assignment afresh, discussed clause by clause and finalized the revised draft for the entire Guidelines, including live load analysis for simply supported spans ranging from 10 m to 75 m in increments of 5 m in span length, with 1-lane to 4-Lane carriageway, showing B.M. at mid span and S.F. next to support, as per IRC Loadings, GVW loading moving case and GVW crowded case. These were prepared and presented in the form of Graphs and Tables by a Sub-group comprising of Convenor Shri A.D.Narain, Members S/Shri S.G.Joglekar, D.K.Kanhere & Dr. B.P.Bagish and S.Rastogi (as invitee). The Committee after due consideration has recommended the finalized Guidelines for its placement before the Bridges Specifications and Standards Committee and Council.

1.5 The revised draft was approved by B.S.S. Committee in its meeting held on 01.5.2010 and the Executive Committee in its meeting held on 10.5.2010 authorized Secretary General IRC to place the same before the Council. The revised document was approved by the IRC Council in its meeting held on 22.5.2010 at Munnar (Kerala) for printing.

2 SCOPE

2.1 These guidelines and suggested methods are applicable for all types of bridges which are covered by IRC bridge codes, except for the old steel bridges where materials used are different from those being used now and where the existing strength of connections between members affected by deterioration or by fatigue cannot be established by any rational method, as also timber bridges.

2.2 The objective of the guidelines is to establish a common procedure for assessing the strength, evaluating the safe load carrying capacity and provide information about rating and posting of bridge to owners, the traffic control authorities and users and the army officers in-charge of movement of military vehicles. Guidelines also include recommendations for dealing with over-dimensioned and overweight vehicles. The terms rating, posting and over dimensioned/over weight vehicles have the following meaning:

a) Rating of a bridge:

The safe permissible load carrying capacity of the bridges in terms of standard IRC loadings.

b) Posting of a bridge:

The limitations of dimensions and weight of vehicles, axle loads or train of vehicles commonly used in the country, which can be permitted to ply without requiring specific permission or escorted by traffic controllers belonging to concerned authorities.

c) Over Dimensioned/Over Weight Vehicles

Vehicles carrying consignments resulting in any of the dimensions of loaded vehicle in height, width and length being exceeded over the legal limits (section 4.3) are termed as Over Dimensioned Consignment (ODC). Overweight Consignments (OWC) are those carrying total loads beyond 100 Tons. Special multi-axle vehicles (**Item 9 of Table 2**) are also included in this category.

2.3 Rating and posting of a bridge are desirable for all old and new bridges. It becomes essential to do so when:

a) The design live load is less than that of the heaviest statutory commercial vehicle plying or likely to ply on the bridge

b) The design live load is not known

c) Where records and drawings are not available

d) The bridge, during inspections (routine or special), is found to indicate distress of serious nature leading to doubts about its structural and/or functional adequacy

2.4 The rating and posting of a bridge is a complex procedure involving subjective decisions in certain cases. As such, it needs to be carried out by bridge engineers with adequate experience and/or knowledge on the subject.

3 ASSESSMENT OF CONDITION OF BRIDGE

3.1 General

3.1.1 *Inspection and maintenance*

i) Routine inspection of newly constructed bridge will lead to normal maintenance, of which full record should be documented. Any signs of distress should be noted and brought to the attention of concerned authorities for initiating further action. This action can lead to rating and posting exercise.

ii) For old bridges which had not been thus maintained from the beginning shall be inspected in detail and condition survey and investigation shall be carried out. This may lead to minor repair work or major repair/rehabilitation. Rating and posting activities shall take cognizance of the extent of repair work or major repair/rehabilitation.

3.1.2 *Assessment of condition*

Detailed guidelines and a strong data base are essential in order to make a scientific assessment of the condition of the bridge. IRC has published following guidelines to which reference may be made:

i)	IRC:SP:35-1990	Guidelines for Inspection and Maintenance of Bridges
ii)	IRC:SP:40-1993	Guidelines on Techniques for Strengthening and Rehabilitation of Bridges
iii)	IRC:SP:51-1999	Guidelines for Load Testing of Bridges.
iv)	IRC:SP:52-1999	Bridge Inspector's Reference Manual
v)	IRC:SP:60-2002	An Approach Document for Assessment of Remaining Life of Concrete Bridges
vi)	IRC:SP:74-2007	Guidelines for Repair and Rehabilitation of Steel Bridges
vii)	IRC:SP:78-2008	Specifications for Mix Seal Surfacing (MSS) Close-Graded Premix Surfacing (CGPS)
viii)	IRC:SP:80-2008	Guidelines for Corrosion Protection, Monitoring and Remedial Measures for Concrete Bridge Structures
ix)	Special Report 17	State of Art: Non-Destructive Testing Techniques of Concrete Bridges

Where there is a reliable and complete documentation on the design and construction of the bridge, field investigations will be oriented primarily towards identifying the effect of any deterioration, damage or settlement that has taken place. Where such documentation is lacking then in addition to the above field investigations, dimensions

of all the structural members should be taken to prepare a complete set of as-built drawings showing the geometric dimensions only. However, where details of the reinforcement and pre-stressing cables cannot be ascertained to a degree of accuracy required for preparation of as-built drawings or for structural calculations, it will become necessary to resort to intrusive investigations at few critical sections combined with desk study using parametric variation of the likely ranges of the estimated data, including error estimation of the capacity thus calculated.

For all these bridges identified for detailed investigation, field and laboratory testing may be required to an extent, which would depend on the degree of deterioration of the structure.

The present guidelines provide the assessment of the load carrying capacity of the bridge keeping in view its structural scheme and behaviour.

Deficiency regarding the hydraulic parameters of the bridges also need to be assessed for which guidance should be taken from recorded and observed data, local enquiry, changes in the hydraulics of the river and the hydrology of the river basin. For this purpose, a separate study need to be undertaken as per relevant guidelines.

Observations of local scour, erosion/deposition of bed material and changes in local stream flows should also be taken into consideration for evaluating the safety of foundations and efficacy of protective measures.

3.2 Data Needed for Assessment

3.2.1 *Collection of data*

The following documents/data are to be procured to the extent available:

i) IRC Code, Specifications as applicable

ii) Contract drawings updated to reflect as built details

iii) Design calculations

iv) Site records of construction

v) Soil investigation data before and during construction

vi) Material test and load test data

vii) Contract specifications

viii) Post-construction inspection and maintenance reports

ix) Details of all repairs/strengthening work carried out till the date of investigations

x) Hydrological, seismic and environmental data including changes if any (revision of zone for seismic classification and retro-fitting requirements as needed, and seismic retrofitting details, if carried out.)

xi) Prevalent commercial vehicular loads plying on the bridge

xii) Other natural hazards identified, if any.

xiii) Traffic survey data

3.2.2 *Structural condition*

Assessment of structural condition of bridge will take account of the following information which has to be collected during the detailed field investigation:

i) Cracking, spalling, honeycombing, leaching, loss of material or lamination of concrete members in superstructure, sub-structure and foundations.

ii) Corrosion of rebars, exposure of rebars, corrosion in prestressing cables and structural steel members

iii) In-situ strength of materials

iv) Effectiveness and condition of structural joints viz. bolted, riveted and welded connections for steel bridges

v) Conditions of expansion joints, bearings and articulations hinges

vi) Settlement, deformation or rotation producing redistribution of stress or instability of the structure.

vii) Any possible movements of piers, abutments, skew backs, retaining walls, anchorages and any settlement of protective works and foundations

viii) Hydraulic data covering scour, HFL, afflux, erosion at abutments variation, if any, in ground water table and discharging arising out of new irrigation projects or any other reason

The list is not comprehensive but includes majority of factors likely to influence load carrying capacity of the bridge.

3.3 Preliminary Assessment

Preliminary assessment of the structural condition can be made by observing for visible deterioration in the form of extensive cracking, spalling of concrete, large deflections and excessive vibrations. In such distressed bridges, there will normally be time for a preliminary assessment of the distress and its likely effect on load carrying capacity.

For proper assessment of the movements of foundation/substructure, the vertical profile survey should be conducted at the deck level both on the upstream and downstream sides of the carriageway and plotted on a suitable scale. This may be carried out preferably over a reasonable period say six months, for which data be recorded. Once a month and profiles compared in order to detect indication of increase in deflection or any unusual break in the profile. This has been discussed further under sub-clause 3.8 hereinafter. The movement of the expansion joints (both horizontal and vertical) should, likewise, be monitored from time to time.

A study of drawings and calculations (where available or prepared by the rating engineer based on site measurements) together with an in-situ inspection would generally give indication whether the structural component has been overloaded or where reserves are still available. Where necessary, immediate measures should be taken to complete the detailed assessment and decide upon the various options available e.g. derating, closure, replacement, repair, strengthening or no action.

The decision to go for detailed investigation may be taken on the basis of preliminary investigation of some of the spans and/or on the basis of visual inspection alone.

3.4 Detailed Assessment

The detailed structural assessment should include a careful inspection of full bridge using techniques appropriate to the kind of deterioration or damage. Since all structural inadequacies that adversely affect strength or serviceability arise from:

a) Deficiencies within the structure i.e. faults in design or detailing, material or workmanship

b) Change in external circumstances e.g. increase in traffic loading, environmental influences etc. resulting in excessive demands on the structure.

A systematic approach to the structural assessment must include the following:

i) Visual inspection of the structure - this should be carried out in order to detect all symptoms of damage and defects and should include a check on the actual dimensions of the structural element concerned.

ii) Study of existing documents - this should include all the documents as mentioned under para 3.2 hereinabove.

iii) Mapping of cracking pattern in the structural components. All visible cracks should be mapped, with cracks of width equal or more than 0.20 mm and duly recorded.

iv) Assessment of behaviour of the structure under dynamic loading e.g. excessive vibrations and amplitude.

v) Environmental influences - this should include effect of aggressive agents in the atmosphere, ground, soil and effluents discharged in the river as well as effects of temperature, rain, snowfall and seismicity at the location.

vi) Material properties of steel and concrete - several inspection and testing techniques and types of equipment required have been described in subsequent part of this document.

vii) Estimate of loads - the prevalent heaviest commercial vehicular load plying on the bridges and the extent of traffic congestion during peak hours including the traffic mix should be studied in detail. For permitting overdimensioned and overweight vehicles, refer to the provisions under Clause **11**.

3.5 Techniques of Inspection and Testing

The inspection procedures to be followed, a simplified Bridge Inventory Form standard tools for preliminary assessment and the assessment methods including destructive and non-destructive tests have been covered in IRC:SP:35.

State of the Art Methods of Non-Destructive Testing of Bridge Structures should also be studied in this respect to take benefit of the development in this newly developing technology. The possible assessment methods and tests for such cases have been indicated in IRC:SP:35, IRC:SP:74 and IRC:SP:80.

However, all the testing methods are not essential for the assessment. Selection of tests may be made based on the specific requirement of the structure and distress condition. Further reliability of the results in ascertaining the exact extent of distress in the structure should also be given due weightage while drawing conclusions from the same.

3.6 Assessment of Strength of Materials

In-situ testing at selected locations normally produce results with some degree of divergence due to variability of materials, methods of sampling, inherent dispersion in co-relation of measured properties to the desired value of target properties (e.g. rebound hammer test measures surface hardness, and strength is indirectly derived there from).

Usually, it would be necessary to establish upper and lower probability limits for the material properties under examination obtained by use of such indirect methods. Cutting of samples for assessing the material strength of concrete or steel members should be carried out only when essential, as such sampling may entail some risks to the structure. Samples cut from steel structures may lead to fatigue weakness while cores drilled in weak concrete members may act as crack inducers. Therefore, such work should be carried out under close supervision and only after obtaining approval from the Design Engineer with respect to the location and details of proposed sampling.

3.7 Sectional Areas of Structural Members and Location of Reinforcement and Tendons

When reinforcement details are not known, position of reinforcement close to the surface may be determined by covermeter (electromagnetic reinforcement detectors) or making incision at selected locations, taking care not to endanger the safety of the structure.

This equipment would also give an approximate indication of bar sizes and spacing. For reinforcement at depth greater than 120 mm, it will be necessary to use other methods such as radiography subject to availability of such equipment, although this will prove to be expensive due to use of radiographic films.

In prestressed concrete structures, size of tendons can be determined if the end anchorages are accessible. Otherwise radiographic methods can be used. However such methods are not reliable for ducts containing several tendons since individual tendons are difficult to distinguish clearly.

In case radiography is to be used, it shall be carried out only be specialists licensed to handle radio-active isotopes and all health and safety regulations should be strictly observed.

Invasive investigations at few locations by cutting out a chase in cover region (e.g. at mid-span bottom and side of bulbs, or vertical groove in web areas at one or two locations). This will also help to establish the cable-profile in vertical planes.

3.8 Settlement, Deformation or Rotation of Structural Members

A survey should be carried out on the deck along the bridge centre line and on either ends of carriageway and the profile plotted to reveal any untoward sag or kink. Levels shall be taken at intervals of about 5 meters; levels should also be taken on top of

each pier cap at the four corners in order to determine any differential settlement of foundations.

Distortion or buckling in steel components should be carefully investigated as this would result in reduction of their load carrying capacity. Measurement will be made by means of a straight edge or dial gauge to give an accuracy of ±0.5 mm in one metre.

3.9 Full Scale Load Test on Bridges

This has been dealt with separately in Section 8 Reference is also made to IRC: SP:51.

4 TRAFFIC FACTORS

4.1 Bridge design standards and specifications determine, in principle, the load carrying capacity of bridge ensuring that it can safely carry the anticipated motorized vehicular traffic. The design live loads are standardized trains of axle loads or other equivalent loads which lead to load effects at various sections covering and enveloping the effects of actual loads. This allows the bridges to support such traffic with the required factor of safety.

The Motor Vehicles Act and regulations, limit the axle load the Gross Vehicle Weight (GVW) and impose a number of dimensional limits on actual vehicles.

It is therefore essential to review existing regulations particularly with regard to the freight vehicles in order to define the actual live load pattern for the existing bridges and to establish an approximate correlation with the standardized design live loads as specified in IRC:6.

4.2 Review of Axle and Vehicle Weights

4.2.1 *Classification by motor vehicles act*

Motor Vehicles Act 1988 classifies motor vehicles into (i) Motor Cycle (ii) Light Motor Vehicle and (iii) Medium & Heavy Motor Vehicles. However for purpose of bridge design, the first two types of loads which may be predominant in urban/city areas are not important. In third type also, the passenger vehicles are lightly loaded as compared to the commercial vehicles. Hence this guideline covers only commercial vehicles with GVW more than 16 Tons.

Maximum safe axle loads specified in Motor Vehicles Act (MVA) are further augmented by data taken from manufacturers for special purpose vehicles plying on Indian roads. This is produced below in **Table 1 and 2**.

Table 1 Safeaxle Loads

Description of Axle	Weight in Tonnes	Remarks
Single axle fitted with 1 tyre	3.0	As per MVA
Single axle fitted with 2 tyres	6.0	
Single axle fitted with 4 tyres	10.2	
Tandem axle fitted with 8 tyres	19.0	
Tandem axle fitted with 12 tyres	24.0	
Multi-axle fitted with 4 tyres on each axle	10.2 (on each axle)	Non-standard vehicle
All other special purpose carriers with different type arrangements and axle loads, or with axle loads higher than 10.2 tons.	X	"

The laden weight of vehicle, including multi-axle vehicles, must not be more than the sum total of all maximum safe axle weights. Thus for the four axle semi-articulated vehicle shown at item Sr. no.6 - **Table 2**, comprising of tractor with two tyres on front axle and four tyres on the rear axle, and trailor having tandem axle with eight tyres on the rear could carry maximum of 35.2 tonnes, (6+10.2+19 tonnes).

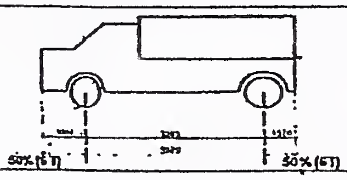
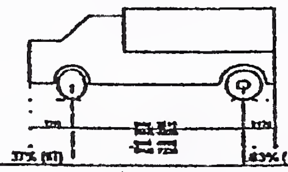
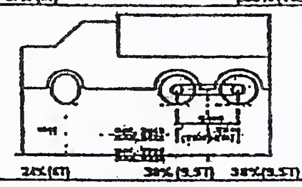
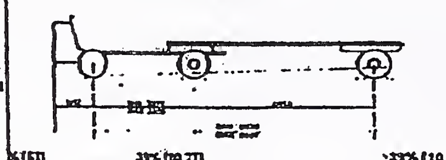
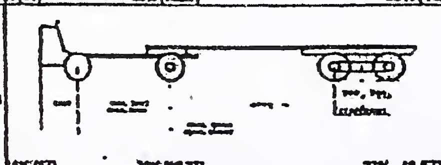
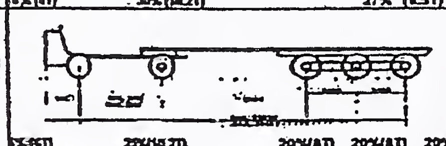
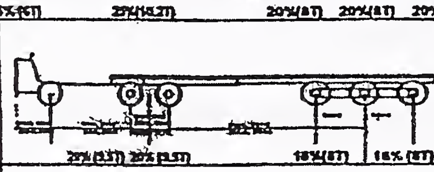
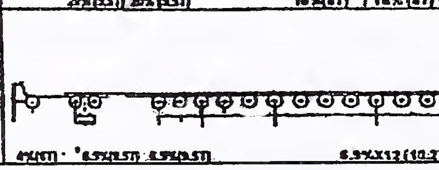
4.2.2 *Commercial vehicles operating in India*

The vehicles commonly plying on Indian roads shown in **Table 2** giving types of vehicles axle position, axle weight and GVW.

It is possible for a specific bridge location to have different and heavier traffic due to nature of local industry, predominance of certain type of heavy vehicles (e.g. for mining industry, petroleum industry, steel mills, etc.). Hence for such location it is recommended that special surveys be conducted to determine actual loading to which the bridge is subjected, as per IRC:5 - General Features of Design.

Table 2 Classification of Commercial Vehicles

CLASSIFICATION OF COMMERCIAL VEHICLES

Sr. No	Type Description	Type No.	Total No. of axles	Axle / Tyre arrangement: Tractor, Front End	Axle / Tyre arrangement: Tractor, Rear End	Axle / Tyre arrangement: Trailor, Rear End	Configuration & distribution of total load (Nominal GVW)	Nominal GVW (Tonne)	Recommended Overload Factor	Standard Deviation
1	LIGHT VEHICLE	1					TWO WHEELERS, THREE WHEELERS PASSENGER AND GOODS VEHICLES			
2	RIGID LIGHT COMMERCIAL VEHICLE	2A	2	Axle: 1 Tyres: 2	Axle: 1 Tyres: 2	—		12.0	1.4	0.32
3	RIGID MEDIUM COMMERCIAL VEHICLE	2B	2	Axle: 1 Tyres: 2	Axle: 1 Tyres: 4	—		16.2	1.4	–
4	RIGID HEAVY COMMERCIAL VEHICLE	3A	3	Axle: 1 Tyres: 2	Axle: 2 Tyres: 8	—		25	1.4	0.29
5	ARTICULATED HEAVY VEHICLE (2 AXLE TRACTOR+1 AXLE TRAILOR)	3B	3	Axle: 1 Tyres: 2	Axle: 1 Tyres: 4	Axle: 1 Tyres: 4		26.4	1.4	0.32
6	ARTICULATED HEAVY VEHICLE (2 AXLE TRACTOR+2AXLE TRAILOR)	4	4	Axle: 1 Tyres: 2	Axle: 1 Tyres: 4	Axle: 2 Tyres: 8		35.2	1.4	0.55
7	ARTICULATED HEAVY VEHICLE (2 AXLE TRACTOR+3AXLE TRAILOR)	5	5	Axle: 1 Tyres: 2	Axle: 1 Tyres: 4	Axle: 3 Tyres: 12		40.2	1.4	–
8	ARTICULATED HEAVY VEHICLE (3 AXLE TRACTOR+3 AXLE TRAILOR)	6	6	Axle: 1 Tyres: 2	Axle: 2 Tyres: 8	Axle: 3 Tyres: 12		49	1.4	0.28
9	MULTIAXLE HEAVY VEHICLE (Example)	Multiaxle	15	Axle: 1 Tyres: 2	Axle: 2 Tyres: 8	Multiaxle 4 tyres each no. of axle (depending on design weight carried)		147.4	Not applicable	–

Note :

1. For Transverse spacing and types refer Fig. 2
2. Recommend overload factors are based on recent surveys carried around in different cities on some stretches of National Highway/State Highway and are the mean overload factors. These can be modified as per local conditions.
3. Overload factor for design checks can be taken as recommended in Table 2 for long span bridges likely to carry more than one GVW class vehicle under consideration for maximum live load effect. For small spans carrying one vehicle, the LL/ DL ratio is high and effect of LL should be checked for overload factor of (1.4 + 1.65 x standard deviation).

4.3 Vehicle Dimensions

a) Height

According to the MVA 1988, the maximum height of vehicle other than a double-decker is 3.8 metres unless it is carrying an ISO series 1 Freight Container, in which case the height must not exceed 4.3 metres. A double-decker vehicle must not exceed 4.75 metres.

b) Width

A public service vehicle or transport vehicle other than a motor cab, must not exceed 2.5 metres in width.

c) Length

A rigid truck must not exceed 11 metres. On routes or in areas approved by the State Government, buses may go upto 12 metres in length. Articulated vehicles must not exceed 16 metres whereas trucks/trailer combinations have a maximum limit of 18 metres.

4.4 Speed Limit and Overload Factor

4.4.1 *Speed limits*

The general speed limits on National and State Highways as defined by MVA for various types of vehicles are given below. For the full list reference may be made to MVA.

- Medium/heavy goods vehicle (Rigid) : 65 km/hr
- Medium goods vehicle with not more than one trailer or heavy articulated goods vehicle : 50 km/hr
- Heavy goods vehicle with not more than one trailer : 40 km/hr

4.4.2 *Overload factors*

Industrially manufactured vehicles generally follow limitations of gross vehicle weight. The manufacture of trailers being wide spread and is more difficult to control and hence many types of trailers ply the roads. With increasing cost of fuel, the tendency of transport operators is to increase freight weight. The over-loading has been observed and measured in a number of surveys on the basis of which the observed overload factors are indicated in **Table 2**. It is useful to note that:

a) Empty (returning) vehicles have not been counted in the survey

b) The overload factor is calculated as actual total gross weight divided

by theoretical maximum gross weight which is based upon the number of axles and tyre configuration. The mean overload factor based on the data from few surveys is indicated in **Table 2**.

c) The maximum over load factor is not to be taken as the absolute largest observed overload factor but is an upper fractile above which only 5 percent observations may fall. It is calculated as max. = (mean +1.65 standard deviation). Any other fractile can be calculated on the basis of assumed statistical distribution of overload factors.

However, as discussed above under special circumstances, the actual upper fractile observed in specially conducted survey may be used to avoid damage to bridge.

4.4.3 *Over dimensioned and overweight vehicles*

Over dimensioned and overweight vehicles need separate treatment as discussed in Section 11.

4.5 Special Traffic Surveys

Present day traffic is estimated to contain a substantial portion of freight vehicles particularly on National or State Highways and those having port connectivity Information on the traffic composition can be obtained through traffic surveys at bridge approach.

If such survey data are not available, it may be necessary to carry out special traffic studies in the following manner:

a) Manual vehicle counts (per hour and category, per traffic direction) during specified periods of day or night.

b) Manual counts and static weighing of a small sample of vehicles (10 percent of vehicles of any particular category)

c) Counts and automatic measuring of axle loads during a specified time period in addition to measuring axle spacing and sequence.

The last type of survey involves sophisticated measuring equipment, but enables determination of statistical distribution of axle loads and other parameters such as vehicle speeds, spacing between vehicles, spacing between axles of the same vehicle, etc.

5 RATING AND POSTING METHODOLOGY - GENERAL

5.1 Rating Methods

The following three methods may be followed for rating of bridge structures using IRC:6. Loading Standards and relevant codes covering the type of bridge under review.

i) **Analytical Method** is applicable when the as-built or contract drawings and specifications followed are available, or when such drawings can be prepared by site measurement to an acceptable level of accuracy (e.g. for steel, masonry or composite bridges). In any case correctness of the available drawings shall be verified at site, since quite often "as-built" drawings/data is not available. This method is covered in Section 6 and Section 7.

ii) **Load Testing Method** is suitable when no construction drawings and specifications originally followed are available, or when data for design cannot be obtained from reference to literature and the condition survey, and/or when the extent of corrosion and loss of strength cannot be assessed during condition survey. Guidelines for this method have been provided in Section 8.

iii) **Correlation Method** - In certain cases, it is possible to ascertain the safe carrying capacity of the bridge structure by correlating the sectional details of the structure with those bridges having identical specifications and sectional details and whose safe load carrying capacities are known.

Even in this case, it is necessary to know the differences and relative deterioration of the structures under review vis-a-vis the details of those structures whose safe carrying capacities are known, so that proper assessment by correlation and factoring the known differences and relative deterioration can be made. This presumes, however, that the physical condition of the bridge under review is otherwise satisfactory. The method is not further discussed in these guidelines.

5.2 Rating of Recently Built and New Bridges

The rating methodology for existing bridges will be that the bridges would be rated for standard IRC live loads as specified in IRC:6 (Section II), and/or for the modified loading suitable for local conditions as directed by IRC:5. Therefore, for new bridges and un-deteriorated recently built bridges, the highest class of design live load as envisaged in the original design shall be taken as the load for rating of the bridge.

Thus the description will be rated for highest loading of Class-AA, Class-A, Class-B or one of the classes such as 70R, 40R etc., as described in Appendix-I of IRC:6 (Section II). The requirements of the design codes covering the type of bridge under review have to be fully satisfied.

In case of recently built bridges, some local defects such as honeycombing, spalling of concrete, onset of corrosion, local deep scour etc. may become apparent during routine and/or special inspection. Such defects can be taken care of by localized repair, and it is not necessary to re-evaluate the rating of bridge due to the same. It should be realized that all codes have built-in margins for time dependent loss of strength, and for localized reduction of strength at sections, which are not critical. These are not explicitly stated in the codes. However, local reduction up to 10 percent estimated on the basis of calculations need not call for re-rating of the entire bridge.

5.3 Rating or Re-rating of Old Bridges

Rating of old bridges for which original designs are not available, and the re-evaluation of previously rated structures in situations where bridge had suffered deterioration of strength of any of the main components from superstructure to foundations, requires careful and detailed evaluation of many complex factors and conditions. Items that need to be included in this evaluation are briefly discussed below:

5.3.1 *Loads*

The loads to be considered in analysis of bridge for rating purposes, are of three types:

i) Design live loads at the time of construction
ii) Design live loads in force at the time of re-evaluation
iii) Changes in other loads in loading standard IRC:6 (e.g. wind and seismic loads)
iv) Changes in other loads based on field observations such as design flood level

5.3.2 *Stresses*

The allowable stresses are to be taken as per the relevant IRC Codes, governing the type of structure under review.

The allowable stresses normally take into account the long terms effects. However, these may need to be downgraded in case of specifically observed deterioration of materials as discussed below.

The changes in the design codes from the time of original design also need to be taken into account.

5.3.3 *Materials*

i) The information about the original material specifications have to be obtained from the records. Knowledge of the year of construction allows reference to be made to codes and specifications which were in force at that time and allows one to make an assessment of materials most likely to have been used and their design strengths considered at that time. If the information is not available, samples of materials taken from the bridge itself can be examined/tested to determine the type and strength of steel, grade of concrete etc. While making assessment of the strength of concrete based on the core samples, many factors need to be taken into account. Reference is made to **Annex 2** for more information on this topic.

ii) The loss of effective section of reinforcement, or prestressing steel has to be based on the investigations specifically made to assess the same. Long term prestressing losses are known to be higher than those estimated using earlier codes. The creep and shrinkage effects are also known to be higher. The original warning about likely higher loss of prestressing force may have come from actual observations during periodic inspection, special inspection or due to unsatisfactory performance such as excessive deflection, vibration etc. It is possible in such cases that the defect may be observed in only a part of the bridge (one or two spans out of many, or a few piers/foundations). In such cases, re-rating need not be based on the strength of the/ weakest span/pier/foundation, but can be carried out after repairing/ strengthening the affected portion so that better overall rating could be maintained. (This approach is termed as improving the strength up to that of the next weakest link after repair).

5.3.4 *Fatigue*

Fatigue is generally relevant for steel bridges. In absence of knowledge of loading history of road bridges, it is difficult to make assessment of fatigue history, or of fatigue cycles expected in future. The only possible assistance for assessing damage by fatigue or assessing the remaining life is laboratory testing of material samples taken from the bridge. Estimation of effect of fatigue on joints and connections is not possible to make, and one has to depend on visual inspection, and observations such

as cracking of welds, loosening of bolts, extent of corrosion etc., for assessing these items.

5.3.5 *Design philosophy*

5.3.5.1 Working load/allowable stress design

Till now (i.e. 2009) IRC Bridge codes are based on the working load/allowable stresses philosophy of design. The loads arising from natural phenomena are taken into account by specifying loads arising from infrequent events, as in case of wind (50 years return period); either of maximum observed flood irrespective of return period, or estimated flood of 50 years return period. Earthquake loads are taken from the IS 1893, which are based on a method explained in foreword of IS1893. Reference to the same is made for details. These loads are combined directly by superposition in load combinations, each combination representing some physical condition of loading for the bridge. However, since publication of IS 1893-2002, the method of evaluation of seismic response has undergone changes, which methods are also adopted by IRC:6. These changes need to be considered in the analysis. For purpose of rating of such structure (i.e. upto 2009) shall be based on the working stress design.

For each of the combination, permissible stresses in materials are stipulated. In case of rare combinations of maximum live loads and design flood, taken together with maximum wind or earthquake, a lower factor of safety may be allowed within elastic limits, taking into account the condition of the bridge (refer IRC:6)

For working stress design, the allowable stresses to be considered will be the higher value of (i) and (ii), but limited to the value prescribed under (iii), as follows :

(i) Allowable stresses considered in original contract documents such as original design calculations and technical specifications.

(ii) As provided in **Annex 1**.

(iii) Allowable stresses obtained from strength tests by field and laboratory investigations.

5.3.5.2 Limit state design philosophy

Limit State Design Philosophy based on defining various "limits" which should not be exceeded by the structure during the expected design life of the structure when exposed to loading from usage, and loads arising from natural environment. The deteriorating effect of aggressive elements of the environment in which the structure is situated are also controlled. The risk of exceeding the limits cannot

generally be made zero, but is kept very low. The risk is assessed in terms of probability that is sufficiently small and acceptable without making the structure uneconomical or unaffordable. Many countries of the world has based their design codes on this approach and IRC has incorporated some of the concepts in their working stress methods indirectly. However new design codes based on limit state philosophy are under preparation. These methods are very useful in assessing the suitability of existing structures in a rational way examining the same in light of the increased risk that can be taken while continuing the use of the structures instead of replacing it at high cost.

The details of this method are covered by the new design codes which are under preparation. Till such time, working stress design as above may be adopted. This is specifically relevant for the assessment of effects of over-dimensioned/overweight vehicles. This is discussed in Section 11.

6 ANALYTICAL METHOD OF BRIDGE RATING

In rating exercise design has to pass all the requirements of design codes. However, while making analysis for posting of bridge, certain deviations can be made in load combinations as discussed in Section 8.

6.1 Carriageway Width

Irrespective of the provisions of original design, the number of lanes to be considered for loading with live load should be based on actual width available in each direction of traffic in case of divided carriageways or total undivided width for two-way traffic. The number of lanes should be as per **Table 2** of IRC:6.

6.2 Loads and Load Combinations for Rating

The loads and load combinations should be as per requirements of IRC:6. For assessment of live load to be used, attention is brought to the changes in design practices over the time, especially with respect of use of Class AA and Class 70R load. Design for both Class AA & 70R, or design for one of Class AA or Class 70R only, or total design for one and checking of superstructure (or deck slab) only for the other may have been adopted in the original design. The design of road slab for girder bridge had been governed by Class 'AA', but while using 70R only for design, the deck-slab may or may not have been checked for Class AA. The method actually used in the original design needs to be established, either by records or through fresh calculations.

6.3 Design Method

The working load allowable stress method shall be used for the computation of rating of existing bridges, except for the bridges designed on limit state design philosophy.

6.4 Logical Sequence of Rating by Analysis

The following sequence of activities will normally be followed:

Activities at Stage-I - Establishing need for re-rating

1) Identify necessity to re-rate the bridge based on field observations, defect reports, change in use, change in design codes or for any other reason.

2) Inspect all components and all spans to establish the types of defects, their severity and rank the defective elements such as foundations, piers, bearings, superstructure and its sub-parts by the severity of defect (Refer IRC:SP:35-1990 "Guidelines for Inspection and Maintenance of Bridges").

3) From the total defects noted under (2), remove those defects which can be covered normal maintenance to establish the remaining defects which are likely to affect the load carrying capacity.

4) From the list as arrived under (3) above, for each of the group of foundation/pier/bearing/superstructure elements identify the critical elements and assess if special repairs/strengthening of the same is likely to raise the overall rating significantly. If so, rate the bridge assuming that the worst elements are repaired.

Activities at Stage-II : Initial Desk Studies

1) Collect all design drawings, calculations, and construction data available with owner department.

2) Look for published literature/reports of the bridge. Often, for important bridges, articles might have been published soon after construction.

3) Prepare a "design basis" report containing original design data, including applicable codes, design loads, materials, etc.

4) Establish the materials and strengths as per original data. If any of the essential information is missing the same has to be collected by methods described in Stage-III.

Activities at Stage-III : Field Investigations

1) Verify by visual inspection and estimate the extent of corrosion by observing "in-place" loss of section. Collect samples for lab investigation, if required, with duecare for passage of traffic ensuring structural safety.

2) Assess quality of concrete by NDT methods. Take concrete core samples for testing at locations of interest. For interpretation of test results, Ref. IRC:SP-74 and IRC:SP-80.

3) Inspect other elements like foundations/piers/bearings/superstructure elements etc., to assess their criticality in influencing rating (i.e. capacity of bridges to carry LL).

4) Properly repair concrete, which has been broken for inspection of steel etc. using non-shrinking material.

Activities at Stage-IV : Desk Study for Fresh Design Assessment

1) Complete the Design Basis Report for re-design based on Stage-II & Stage-III including any changes in loads other than live loads. The superimposed load may have to be increased to take care of additional material of wearing coat to fill up low levels created on the deck by long term creep effects or provision of new surfacing over damaged in-situ concrete wearing coat or for any other reason.

2) Establish structural parameters to be studied.

3) Calculate the actual strength parameters on the basis of strengths and allowable stresses established from the field data at Stage-III and applicable codes.

4) Carry out analysis and establish strength demands using the then IRC:6 design loads. This may have to be done for the desirable level of live load rating (70R, Class A, etc.), and if the same is not satisfied, for the next lower level of design live load.

5) If the assessed strength in (3) above is more than 90 percent of the "desirable" strength decided in step (4), accept the bridge as satisfactory for that class.

6) If not, repeat step (5) for the next lower Class and so on till the results satisfying requirements of that class.

7) For rating as per latest IRC:6, the iterative process may be resorted to, - for arriving at the safe load commensurate with strength worked out as per (3)

6.5 Explanatory Notes for Application of Analytical Method

Computation of Capacity of Superstructure

a) **Detailing of steel in old (existing) bridges**

When fully detailed drawings are available the same should be used. Problem arises when full details are not available and only information at critical sections can be established at Stage-II by non-destructive or semi-destructive investigative techniques. This is illustrated by example of solid slab superstructure where magnetic covermeter will locate spacing of bars in a non-destructive way and diameters can be confirmed by partially chipping off cover and exposing the bars. The extrapolation of this knowledge to other sections has to be based on the knowledge of practices followed in the period when construction was carried out, regarding curtailment practices, minimum distribution steel, detailing of corners in case of skew bridges, etc. This can be established by comparison with detailing of similar structures constructed by the owner department in that period.

b) **Use of Analytical Methods**

While assessing the strength that might have been built-in in various sub-elements, or parts of the superstructure it is useful to analyse it by using the same methods of analysis as prevalently used in that period. For example use of Courbon's method for girder bridges, or use of Guyon-Massienet method for slab bridge or use of equivalent plate for wide, bridge where Courbon's method could not be used. This will yield more useful information about the distribution of load on the existing superstructures than using modern computerized methods.

The new grillage analysis, FEM methods or other accurate methods of analysis can be (or should be) used to get more accurate distribution of bending moments, axial forces and shear forces caused by the live loads and comparing the same with the distribution of existing strength as assessed by use of old methods.

c) **Slab Bridges**

For determination of strength of old or deteriorated slab bridges, more accurate analysis should be carried out. **Table-3** can be used to arrive at the safe axle loads at primary evaluation.

Table 3 Safe Axle Load for Rcc Slab Bridges

Effective Span (m)	Thickness of Slab (mm)	Safe Axle Load (T)
2	150	**9.5**
	175	14.5
	200	21.0
3	200	11.5
	225	15.5
	250	20.0
	275	25.5
4	225	**9.5**
	250	13.0
	275	17.0
	300	21.5
5	250	**9.0**
	'275	11.5
	300	15.0
	325	19.0
	350	23.0

Effective Span (m)	Thickness of Slab (mm)	Safe Axle Load (T)
6	300	10.0
	325	13.0
	350	16.0
	375	19.0
	400	24.0
7	325	9.0
	350	11.5
	375	14.0
	400	17.5
	425	21.0
8	375	9.0
	400	12.0
	425	15.0
	450	18.0
	475	21.5

Notes:

i) Slab thickness includes a cover of 25 mm

ii) A 75 mm thick wearing coat is assumed over the slab

iii) No separate allowance for impact need be made on the safe axle loads as the same has already been accounted for.

d) Masonry and Plain Concrete Arch Bridges

For these types of structures only the methods used in the relevant period of construction have to be used. These are available from literature and old PWD hand books. Often, the design has been based on the 'rule of thumb' or use of 'standard' sections or nomogram. Use of one such nomogram is shown in **Fig.1** for arches. In this method:

i) The provisional safe axle loads (before applying various factors) for different spans, thickness of arch ring and depth of cushion may directly be read from the nomogram in **Fig.1**.

ii) Assessment arrived at from the nomogram are in terms of a maximum provisional axle load (before applying various factors), which may be taken as the combined load in case of tandem axles,

iii) The allowable axle loads and thereby the rating shall be arrived at from the provisional axle loads obtained above, by multiplying

these loads by appropriate profile factors, material factors, joint factors, support factors, etc., specified in **Annex 2**.

iv) For plain concrete arches, the material factor of 1.5 shall be used in the analysis.

e) **Reinforced/Prestressed/Composite Bridges using Arch as part of Bridge**

These should be analysed by applicable rational methods of design including the arch action.

2. Substructure including Bearings

The overall rating of the bridge is not expected to be affected by bearings.

a) Condition survey of bearing will reveal which bearings will need replacement, and which need maintenance. Also local zones of stress concentration of substructure which are immediately in the vicinity of the bearing can be inspected and identified for repair, if needed.

b) Condition survey of pier caps and piers is important. If these are weakened for any reason, the bridge rating can be affected. This is especially true for old masonry/plain concrete piers. Local repairs, or jacketing, may turnout to be a cheaper and viable option than to "de-rate" the bridge. If this is not the case, the re-rating will have to be done as governed by the strength of substructure.

c) If development of deep scour is the cause of distress - such as excessive stresses or excessive vibrations - it can be tackled by filling the deep scour holes by stone/gabions and general bed protection and re-rating of bridge can be avoided. If, however, the general bed level has also changed, the resultant changes in hydraulics may call for re-rating.

3. Foundation Condition

The distress in foundations is the most difficult item to inspect, or strengthen.

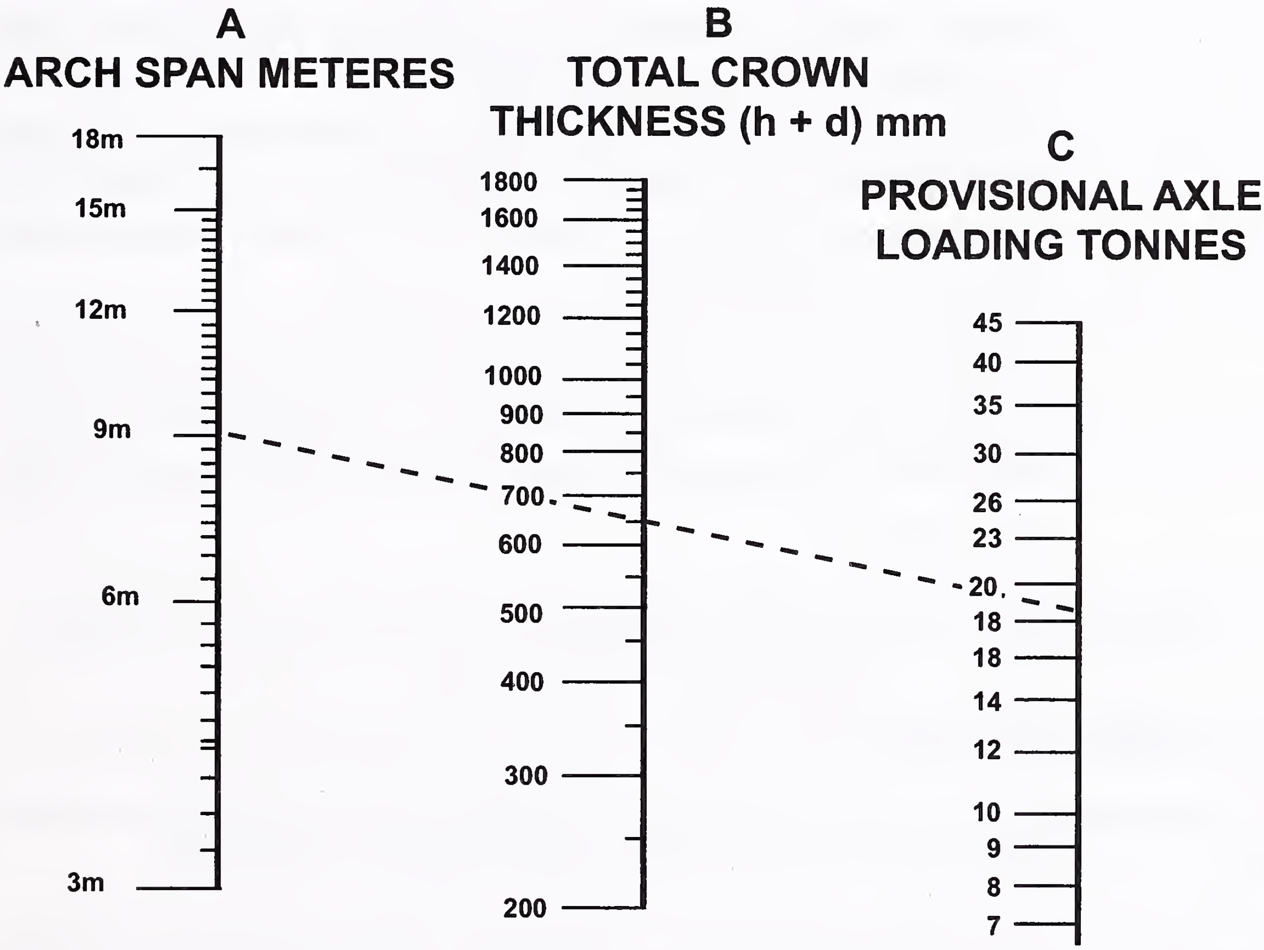
a) Where excessive scour is the source of danger, the same can be treated as discussed in 2(c) above.

b) Where excessive settlements have been observed, one of the two situations may exist.

One is that foundation has stabilised after settlement. This is more likely to happen in open foundations in sandy soil and rocky strata. This is also possible in case of piles where bottom of pile has not been

d.The thickness of ring at Crown

h. The average depth of fill between the road surface and the arch ring at the Crown



Example

Span	= 9 Meters
Span/Rise Ratio	= 4
Span/Rise Factor	=1.0
Shape Factor	= 0.8
Profile Factor	= 1 × 0.8 = 0.8
Crown, Thickness d	= 400 mm
Ring Factor	= 1.20
Fill Factor	= 0.90
Fill Depth h	= 250 mm

Width Factor	= 0.90
Depth Factor	= 1.00
Mortar Factor	= 1.00
Joint Factor	= 0.9 x 1 × 1 = 0.90
Support Factor	= 0.90
Aburment Faullt Factor	= 0.80
Reduction Factor For Impact	= 0.90

$$\text{Material Factor} = \frac{1.2 \times 0.4 \times 0.9 \times 0.25}{0.65} = 1.085$$

The porvisional axle loading for an arch. 9 m span with total crown thickness of 650 mm is, from the nomogram 18.7 tonnes

Allowable axle load = 18.7 × 1.085 × 0.95 × 0.90 × 0.80 × 0.9 = 8.95 tonnes

Fig. 1 Nomogram for Determining the Provisional Allowable Axle Loading of Exisiting Masonary Arch Bridges before Applying Factors (to be used only for Rating and not for Design Purposes)

Note: This would mean that the arch under consideration is safe for 12T standard truck

properly cleaned before concreting and pile has settled through this material and then has come to rest on good sound strata.

In these situations, re-setting the bearings and bridge at the original design levels will sort out the problem.

The second type is of foundation in clayey soil which have not reached stable condition and settlements may continue. In such cases, if any foundation treatment is not feasible, the only option is to de-rate the bridge and reduce the loads.

c) When structural damage to the elements of foundation are noticed or suspected (such as corrosion of steel in pile foundations) or damages in piles below ground level revealed by non-destructive methods. Generally it is not possible to repair the same and de-rating of bridge will be called for.

7 ANALYSIS FOR POSTING BASED ON ACTUALLY PLYING VEHICLE POPULATION

7.1 General

The rating of the bridge based on the hypothetical trains of live load cannot be directly used for putting controls/restrictions to traffic actually plying on the bridge.

For this purpose, the maximum effects of loads plying on bridges need to be calculated instead of IRC:6 Live Loads, and combined with loads from other sources (flood, wind, etc.). These effects can then be compared with the 'rating capacities' obtained from Section 6 or with the directly assessed strengths based on actual observations (Section 9).

For this purpose the live loads to be considered are described below:

7.2 Live Loads Plying on the Bridge

1) IRC:5 General features of design permits vide Clause 102.7.1 & 102.7.2 use of 'any specific variation' from IRC:6 clauses to cover special load conditions. Deviation from the standard loading classes is made under sanction of this clause.

2) The data of actually plying vehicles in the country has been presented in **Table 2** of Section 4. The 'design train of vehicle' to be considered is based on the following considerations:

   a) Bridges should be posted for one of the Nominal GVW Classes shown in **Table 2**, except over dimensioned vehicles.

b) For over-dimensioned and overweight consignments carried on multi-axle vehicles special studies have to be made on case to case basis, as discussed in Section 8. However, on specially selected routes such as port connectivity routes or routes serving heavy industrial complexes maximum permitted load can be estimated based on the maximum number of axles that can occupy the critical length (based on influence line diagrams for bending and shear), multiplied by the maximum permitted standard axle load. If the axle load and placement of tyres is different from the standard axles individual case based studies have to be carried out.

c) After selecting the reference GVW class of vehicle, distance between two successive vehicles and the impact factors are to be taken as below:

i) Reference GVW Vehicles at 4.0 m spacing between last axle and first axle of next vehicle without impact factor with appropriate overload factor.

ii) Reference GVW vehicles at 20.0 m spacing between last axle and first axle of next vehicle with impact factor with appropriate overload factor.

The rationale behind these cases is explained in notes below.

i) Moving Traffic Condition

The road on which structure is situated may have been designed for vehicle speed of 80 to 100 km/hr. However, at more normal (lower speed) of about 60 km/hr. the reasonable distance between two successive vehicles is considered as two to four times the length of vehicles (approx. 10 m to 25 m). For simplifying the calculations, on an average distance of 15 m is specified between two vehicles for all classes of GVW. At this distance full impact factor should be considered as per IRC:6.

ii) Crowded Loading or Traffic Jam Condition

Vehicle standing 'bumber-to-bumber' with spacing of 4.0 m between back axle of vehicle to front axle of vehicle behind it is observed in practice. Number of vehicle or part there of that should be taken in design of a section is decided by the influence line diagrams for bending and shear for that section.

- No impact factor should be considered in this crowded loading/traffic jam case.

- The overloads factor to be considered should be based on the actual survey data, but should not be less than the average overload factor indicated in **Table-2**.

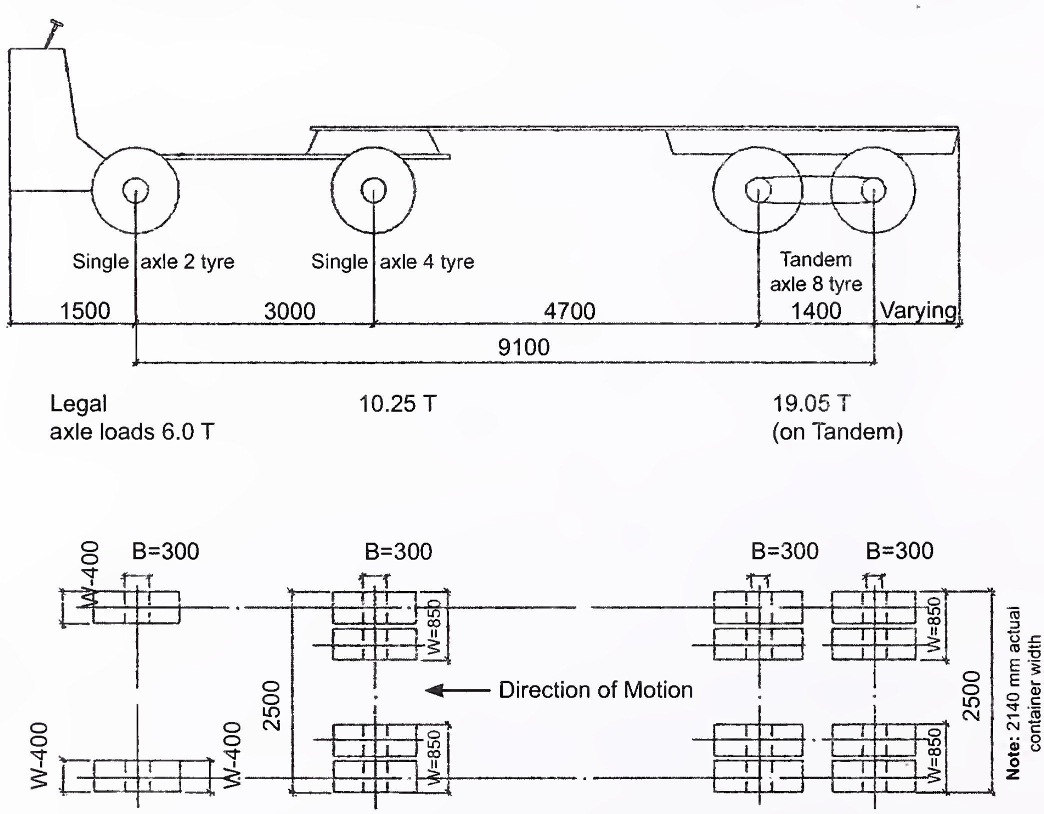
It is important to note that this overload factor is taken only for the purpose of checking structural safety of a bridge, and is not to be construed as an official recognition, or license for overloading vehicles. The laws of the country will govern in these matters.

d) **Transverse Spacing of Vehicles**

The transverse spacing of all vehicles should be taken as per **Fig. 1**, **Fig. 2 & Fig. 3.** The design of road slab supported between webs, or cantilever slab should be verified considering the effect of rear axle loads of the design GVW vehicles Hus impact factor. The concept of effective width as per IRC:21 can be used for the same as a simplified method, in lieu of exact analysis made of full span using grid analysis of finite element method.

For non-standard distribution of tyres on an axle exact analysis taking into account actual position of tyres is recommended.

Load Combinations with other Loads of IRC:6

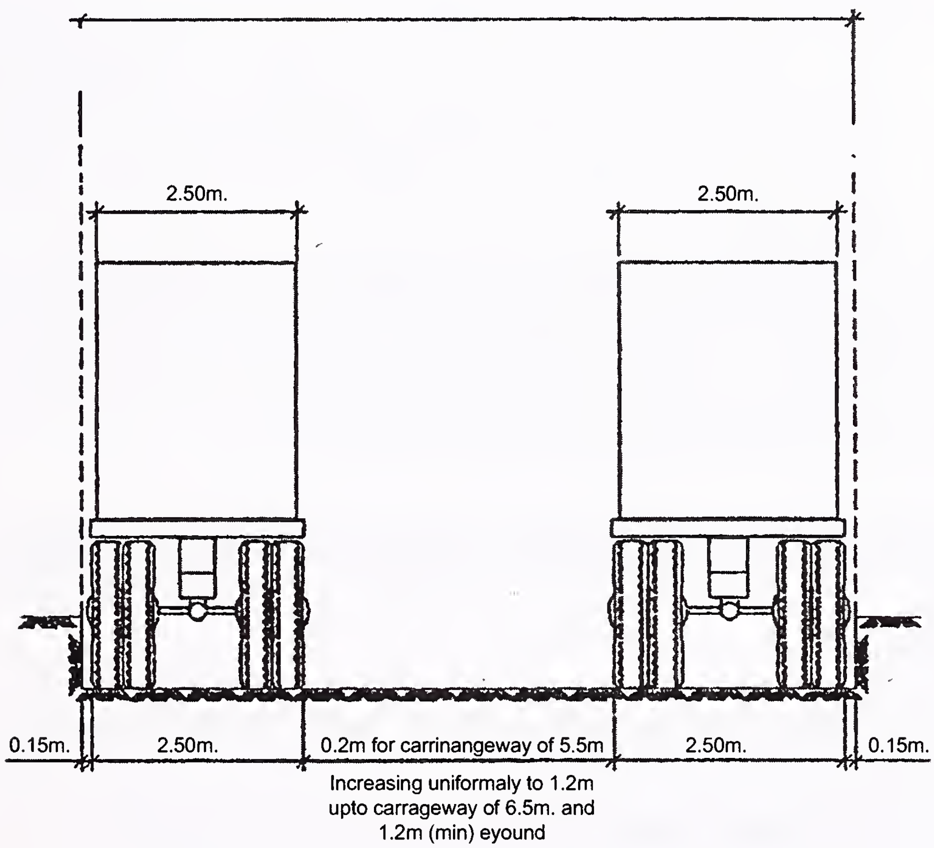


Ground Contact Areas in mm for GVW Type

Note: For IRC loads spacing of Vehicles, location of axles & tyres shall be as per IRC: 6-2000)

Sl. No	Type	'B' Along Traffic	'W' Across Traffic
1	Single 2 tyre spaced at 2.4 m c/c	200	380
2.	Single 4 tyres (for group of 2 tyres)	300	860
3.	Tendons 8 tyre (each group of 2 tyres)	300	510

Fig. 2 Typical Design of Axle Load (Illustrated by GVW 35 Ton)



MINIMUM CLEARANCE OF VEHICLES

Clear carriageway Width	9	f	Remarks
5.5 m to 7.5 m	Uniformly increasing from 0.4 m to 1.2 m	150 mm for all carriageway widths	The clearances as indicated shall apply for other vehicles as well.
Above 7.5 m	1.2 m		

Fig. 3 Minimum Clearance of Vehicles

7.3 Load Combinations with Other Loads of IRC:6

As a general design philosophy, GVW loading replaces only the IRC:6 live loads. As such, other load and load combinations should be used as per IRC:6 and the relevant design codes. Exceptions to above may be considered in the following situations.

i) Where crowded/traffic jam loading is governing, the permissible stresses may be increased for the substructure and foundation design for wind/water current combinations by further 15 percent beyond those allowed by design codes and combination with seismic loads need not be checked.

ii) If the heaviest GVW load which superstructure can carry is considered to be plying very infrequently, the substructure and foundations can be checked on the basis similar to (i) above. Full combination as per IRC:6 and the design code may be made for the frequently plying GVW only.

iii) For old bridges, which are assessed as due for replacement, in the wind and seismic combinations lower values of wind loads reduced upto 50 percent may be used. Seismic checks may be omitted altogether.

iv) For OD/OW vehicles, refer Section 11.

8 LOAD TESTING FOR RATING AND POSTING

8.1 Load Test

Load test for rating purposes and load test for posting purposes are required to be carried out in the following circumstances:

a) Load Test for Rating

i) Load Test for rating is done when it is not possible to determine the rated capacity of a bridge due to lack of essential details as described in Section 6.

ii) For rating of masonry arches load testing is recommended.

b) Load Test for Posting

Load Test for posting is done when details required for verifying the strength of all elements of existing structure by analytical methods is not possible due to lack of reliable data.

8.2 Test Vehicles for Rating

Rating is essentially done to verify which of the Standard IRC loadings described in IRC:6 in combination with other loads can be assigned to the bridge. Use of mobile test vehicles duplicating the axle loads for various classes of loadings should be preferred as compared to the use of equivalent static load which are difficult, time consuming and needs longer closure of traffic on the bridge. The advantage of using such mobile vehicles is that they can be quickly positioned in the exactly required locations. Also being rolling loads, all cross-sections of superstructure are tested without having to workout special loading patterns to represent envelope diagrams of the span for bending and shear. In exceptional cases if commercial vehicles as specified in **Table 2** are used, the number and spacing of such vehicles need to be worked out so as to produce equivalent B.M. and shear at critical sections on those due to the Standard IRC loading.

8.3 Test Vehicles for Posting

The test vehicles will be from amongst those commercially available as specified in **Table-2**. The test vehicle chosen will be the next heavier vehicle than the predominant heavy vehicles presently plying over the bridge. The second next heavier vehicle may be considered for testing, if required, after the load testing with the first vehicle is complete and found to be satisfactory. Use of heavier vehicles, if available, is permitted for testing.

The test vehicle used for purpose of posting shall allow for appropriate overload factor based on actual traffic data in the region.

8.4 Positioning of Test Vehicles

8.4.1 *General*

Test vehicles shall be placed at marked locations on the bridge so as to produce maximum moment effects on girders. While placing the test vehicles at the desired location on the deck, these will preferably be moved from both directions leading to their final positioning.

For posting purposes, the response of the structure to loading may be checked at a few critical selected locations. Usually the following checks may be considered as adequate:

a) Mid-span region and 1/4th span for sagging B.M. for slabs/girders/ box section bridges.

b) Support section for hogging B.M. for cantilever bridges, continuous bridges and bridges with over hangs.

c) Shear at support and at points of changes in web thickness.

8.4.2 *Arch bridges*

For arch bridges, the rear axle of a standard truck shall be placed on the crown and in the case of twin tandem rear axle, the rear twin tandem axles shall be placed symmetrically about the transverse centre line of the bridge.

8.5 Procedure for Load Testing

The test procedure in general shall be as per IRC:SP:51 - "Guidelines for Load Testing of Bridges".

Any requirements arising out of earlier observations about defects/cracking etc., during inspection shall be taken into account.

8.5.1 For concrete girders prior to load testing, observations shall be made for any crack in the structure. The cracks, if any, shall be measured for their width and marked. The external dimensions of the concrete sections and properties of concrete may be used for computation of theoretical deflection.

8.5.2 Prior to testing a whitewash shall be applied at the critical sections for ease of observation of behaviour of cracks and their new formations during the test.

8.5.3 The load test shall be done during such period of the day when the variation in temperature during test is low. Preferably, the testing could be done in early hours of morning or late evening.

8.5.4 The test load shall be applied in stages following the given values 0.5W, 0.75W, 0.90W, 1.0W, where "W" is the gross laden weight of the test vehicle.

8.5.5 For each stage, the correspondingly loaded test vehicle shall be brought to the intended/marked position and observation of deflections shall be made immediately on loading and after five minutes.

The test vehicle should be taken off the bridge and instantaneous deflection recovery and deflection recovery 5 minutes after the removal of the load should be noted.

8.5.6 After the load placement, observation shall also be made for development of any new crack and widening of the existing ones.

8.5.7 Prior to starting of testing, the theoretical deflections for various stages of loading shall be calculated and plotted at points of interest. On this graph, the actually observed deflections shall be plotted during the progress of testing.

The linearity of load deflection should be generally obtained in the test. If two successive readings show excessive deflection of more than 10 percent from the extended linear behaviour, it can be due to onset of non-linear (plastic) behaviour. The test shall be discontinued, temporarily and reference made to the design office for review of the entire procedure. However, deflections shall be continued to be marked for next 24 hours.

Next stage of load increment should be stopped under any of the following conditions:

8.5.8 ***(a) For arch Bridges***

i) Crown deflection or spread of abutment as specified in sub-clause **8.6** is reached.

ii) The recovery of crown deflection or spread of abutment/pier is less than 80 per cent.

iii) Signs of distress in the shape of appearance of visible new cracks a perceptible widening of existing cracks in the arch rib are observed. Methods of measuring crack width have been discussed under Sub-clause **3.5** hereinbefore.

8.6 Acceptance Criteria

8.6.1 *For arch bridges*

Where no crack is observed, the load for rating shall be taken as the least of:

i) The load on rear axle causing a deflection of 1.25 mm in the case of test vehicles having single rear axle and for test vehicles having twin rear axles, the total load on the two rear axles causing a crown deflection of 2.0 mm.

ii) The load causing a spread of abutment/pier of 0.4 mm at spring level.

iii) The load causing recovery of crown deflection or spread of abutment/pier to a value of 80 percent.

The load for rating shall be taken as half the axle load at which a new visible crack or perceptible widening of existing cracks are observed.

8.6.2 *For girder bridges*

The load for rating shall be taken all the least of:

i) The load causing a deflection of 1/1500 of the span in any of the main girders for simply supported spans OR for cantilever spans the load causing a deflection of 1/800 of the cantilever span in any of the main girders. The rotation of pier should be accounted for while calculating the deflection.

ii) The load causing tension cracks of width more than 0.3 mm in any of the girders for normal cases and 0.2 mm for structures exposed to very severe and adverse conditions.

iii) The load causing appearance of visible new diagonal cracks of width more than 0.3 mm for normal cases and 0.2 mm for structures exposed to very severe and adverse conditions or opening/widening of existing cracks close to the supports in concrete girders.

iv) The load at which recovery of deflection on removal of test load is not less than 75 percent for R.C.C. structures, 85 percent for pre-stressed concrete structures. Temperature correction be considered as per provisions contained in IRC:51.

9 BRIDGE POSTING

9.1 General

All postings for bridges shall be made as shown in **Fig.** 4, in terms of equivalent axle loads and/or gross vehicle weights (GVW) of the commercial vehicles plying on Indian roads and satisfying provision of the Motor Vehicle Act as shown in **Table 2**. For overall dimensioned/overweight commercial vehicles the limits shall also be indicated as per Section 11.

9.2 Method of Analytical Computation for Posting

Bridge structure rated for vehicles classes as per IRC: 6 will be posted for the commercial vehicles shown in **Table 2** by comparing the forces caused by GVW vehicles with the design forces imposed by IRC loading.

Annex 3 shows comparison of some of the IRC:6 standard loading, for 1-lane, 2-lane, 3-lane and 4-lane superstructure with simply supported spans from 10 to 75 m vis-à-vis GVW vehicles. Only the main total span moments and shears are presented. These can be used as guidance. The transverse distribution, load and resulting increase in different girders or portions of slab bridges has to be done for each bridge as per its geometry.

9.3 Temporary Traffic Restriction

Depending upon the assessment of the bridge condition. The rating/posting engineer would decide the necessity of the following traffic restrictions on the bridge from safety considerations till the exercise of posting is completed.

i) **Speed Restriction** - to be effective till the detailed investigations and strengthening or rehabilitation work and load testing (if required) on the repaired bridge is complete. The limiting speed of vehicles over the structure will be decided by the bridge authority depending upon the physical condition of the structure.

ii) **Geometrical Restriction** - this would involve curtailing the carriageway width to ensure lesser extent of live load on the bridge at a particular time and/or installation of height barrier on either end approaches to restrict passage of overloaded or oversized commercial vehicle on the bridge.

iii) **Footpath Loading** - depending upon the structural condition of the footpath slab, restriction on load on footpath may be imposed till the distressed part is rehabilitated. Restriction on footpath load may also be; necessary in order to reduce the total load on the bridge superstructure.

9.4 Posting Sign

Postings for bridges shall be made as shown in **Fig. 4**, on both sides of the bridge. The load regulatory and advance warming signs as shown in **Fig. 4**, shall be installed on both sides of the bridge on the approaches from the bridge abutments and at all road junctions leading to the posted bridge.

Load Regulatory Sign

This will be placed at a sufficient distance (not less than 100 m) from the abutment, on both ends of the bridge so that truckers can make arrangements to use detours or to limit their loads to the maximum weight allowed.

Advance Warning Sign

For all bridges to be posted, an advance warning sign indicating a "Load Limit Bridge Ahead" will be placed at least 200 m from the abutments on both ends of the bridge and at all road junctions leading to the posted bridge starting from the earliest major junction.

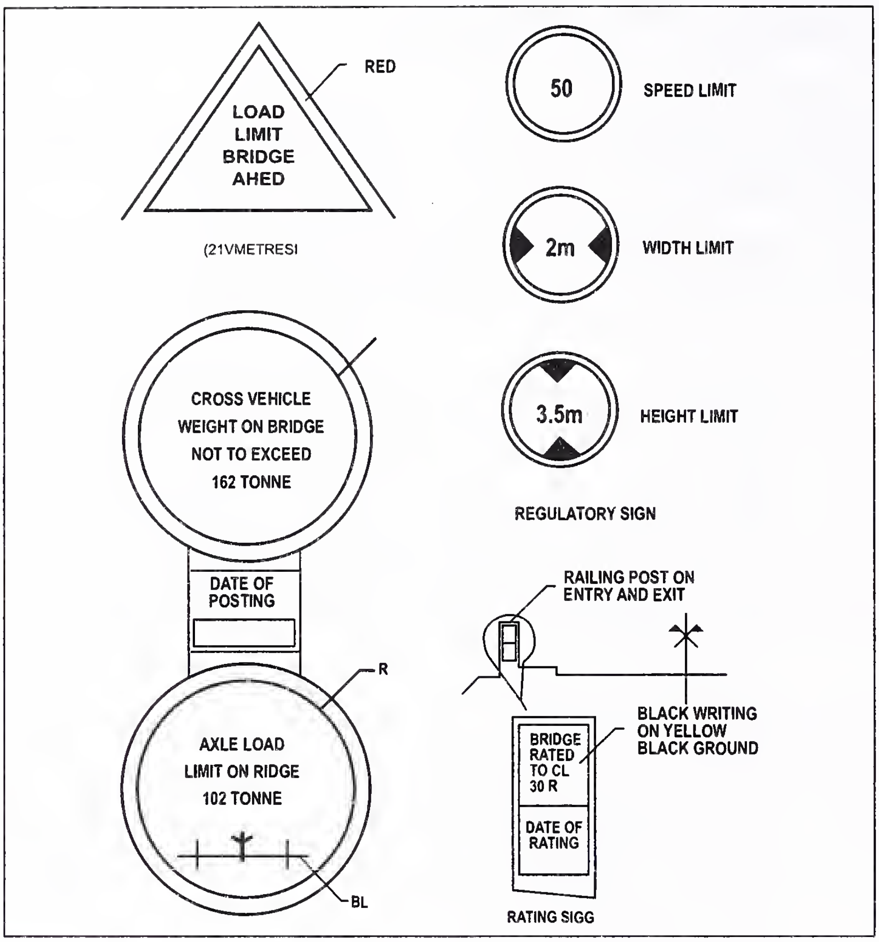


Fig. 4 Bridge Posting Signs Specifications to IRC:67-1977 & SP:31

9.5 Enforcement

9.5.1 Enforcement of restrictions in respect of maximum axle load GVW, speed on bridge and geometrical restrictions may be required for safety of the bridge. This may be ensured by the respective department through the administrative machinery of the State. For bridges of paramount importance (e.g. strategic locations, on highways carrying heavy traffic loads, bridges whose closure will involve very long detour etc.), specialized equipment may be used for such enforcement. These may comprise:

i) Portable or permanent weight bridges or weight-in-motion (WIM) appliances or computerized traffic management systems presently available indigenously.

ii) Doppler Radars for checking vehicle speed on the bridge.

iii) Frame Barriers - suitably designed for specific applications (motorized and remote controlled from a traffic booth; if necessary), such as restricting height/width of vehicles.

iv) Installation of close circuit TV to monitor traffic intensity on the bridge.

9.5.2 The options available to the rating engineer as alternatives to bridge load posting are as follows:

- Restrictions to speed limit
- Restrictions to vehicle dimensions (frame barrier) frequent inspections
- Lane limits
- Repair
- Strengthening

9.5.3 In addition to the posting sign at the distressed bridge site, the following methods may be considered by the enforcing authority for notifying public of the bridge posting to be suitably located at a number of road junctions leading to the posted bridge:

- News release
- Special notice to trucking association legal notice
- Notice pasted at weigh stations
- Weight limit maps or lists

The posted bridges are required to be inspected thoroughly and frequently, at least once a year.

10 REPAIR, STRENGTHENING AND REHABILITATION OF BRIDGES

10.1 Pursuant to the detailed inspection, testing and assessment of the load carrying capacity of an existing distressed bridge, the various options available to the bridge owner and the follow up actions to be taken would be carefully evaluated. Four possible options have been mentioned in IRC:SP:35. One of these options will be to undertake immediate repair strengthening and rehabilitation of the bridge.

The technical scheme for repair and strengthening of distressed bridge would depend on the nature and extent of the distress in the bridge superstructure, substructure and foundations. The objective must not always be to restore the original condition of the bridge. It can be quite sufficient both economically and technically to provide proper strengthening whilst, at the same time, derating the safe load carrying

capacity. In this respect cost analysis can be of help. An estimate can also be obtained by studying the risks involved and giving consideration to the life of the structure as to the success of the repair/strengthening measures proposed.

The above subjects are covered in detail in IRC guidelines SP:40 "Guidelines on Techniques for Strengthening and Rehabilitation of Bridges". IRC:SP:74 – Guidelines for Repair & Rehabilitation of Steel Bridges, IRC:SP:75 – "Guidelines for Retrofitting of Steel Bridges by Prestressing" and IRC:SP-80 – "Guidelines for Corrosion Prevention, Monitoring and Remedial Measures for Concrete Bridge Structures".

11 GUIDELINES FOR PERMITTING OVER-DIMENSIONED/ OVER-WEIGHT VEHICLES

11.1 General

Due to industrialization of many parts of the country, heavy loads much in excess of the capacities of IRC:6 loads and GVW49 T vehicles need to be carried on existing bridges. These loads shall be carried over the bridge with the specific permission of the Authorities. The checking of bridge components to carry such loads and the load combinations as required by IRC:6 need to be reviewed for their suitability by the bridge authority/owner. The following provisions cover the aspects to be reviewed and the possible deviations that can be permitted from the normal design rules.

11.2 Over-Dimensioned Consignments

For these loads, the physical dimensions of the consignment should be such as not to damage any permanent part of the bridges such as handrails, or any structural part for through type bridges. The over-dimensioned vehicles be allowed only with a pilot vehicle and at that time no other vehicle be allowed to ply on the bridge.

11.3 Load & Load Combinations

1) The load carried by over dimensioned and over weight vehicle is certified by manufacturer including the packing, supporting framework etc. The weight of the vehicle itself should be added to the same.

2) The distribution of load on various axles depends upon the rigidity of the trailor and the load itself, apart from the location of load. Some carrier trailors are hydraulically controlled to spread the load equally. If not, loading of trailors should be symmetrical with reference to all

axles. The impact factor may be reduced to 1.0 if speed restriction is placed on it at 5 km to 10 km per hour.

3) The superstructure design should be checked with load travelling at centre with minimum eccentricity of 0.3 m. The permissible over stress on (DL+LL) combination can be taken as 33 percent in R.C.C. bridges. For prestressed superstructures, tensile stresses should not lead to crack widths more than 0.3 mm for structures having non tensioned reinforcement at all sections.

4) For segmentally constructed bridges, without reinforcement across joints, tension should not exceed beyond 2/3rd modulus of rupture, for segments with epoxy joints and no tension for dry joints.

5) Loads should not be transported when wind speed exceeds 40 km/hr. The wind load on the structure and the consignment is to be taken at 5 percent of the design wind speed, with no further increase in the allowable stresses.

6) Seismic loads need not be taken into account.

7) Water current forces should be considered as applicable at time of transportation.

8) Allow able foundation pressure should not exceed more than 25 percent.

9) Provision of temporary supports taken from river bed are not recommended since the load shared between such supports and the superstructure cannot be reliably calculated.

10) If the bridge structure is not sufficiently strong, the alternative methods of transportation should be adopted. These include:

    a) change of route.
    b) Provision of temporary road at river bed and temporary pipe culverts for waterway in case of dry season and shallow depth of water.
    c) Transportation by barges for deep water rivers.

Annex 1
(Clause **5.3.5.1**)

PERMISSIBLE STRESSES IN DIFFERENT MATERIALS

Where working stress method of analysis is done, the permissible stresses in different materials shall be as under :

(i) In structural steel and mild steel, 45 percent extra shall be allowed over the values specified in relevant IRC Standard Specifications and Codes of Practices for Road Bridges.

(ii) In concrete and in masonry, 33.3 percent shall be allowed over the values, specified in relevant IRC Standard Specifications and Codes of Practices for Road Bridges and Design Criteria.

Annex 2
(Clause **5.3.3**)

FACTORS FOR RATING MASONRY ARCH BRIDGES

A. Profile Factors

The profile factor of an arch. F_p shall be arrived at from the expression

$$F_p = F\,S_r \times F_s$$

Where FS_r - the span/rise factor and F_s - the shape factor, shall be as given in **Table 4** and **Fig. 5 & 6**.

Table - 4

Sl. No.	Span/Rise Ratio	Span/Rise Factor (FS_r)	Remarks
1	For L/R upto 4 For L/R over 4 obtain factor from Fig. 10	1.0 1.0 to 0.6	For a given load, flat arches are weaker than those of steeper profile although an arch with a very large rise may fail due to the crown acting as a smaller flatter arch.

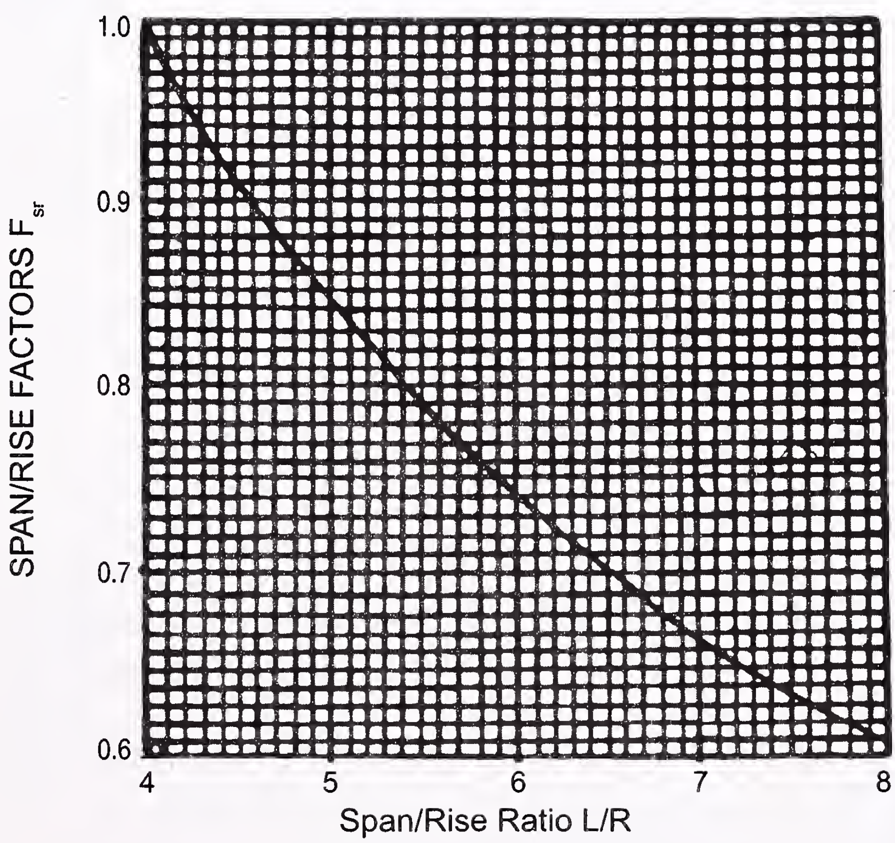


Fig. 5 Span rise factors for masonry arch bridges

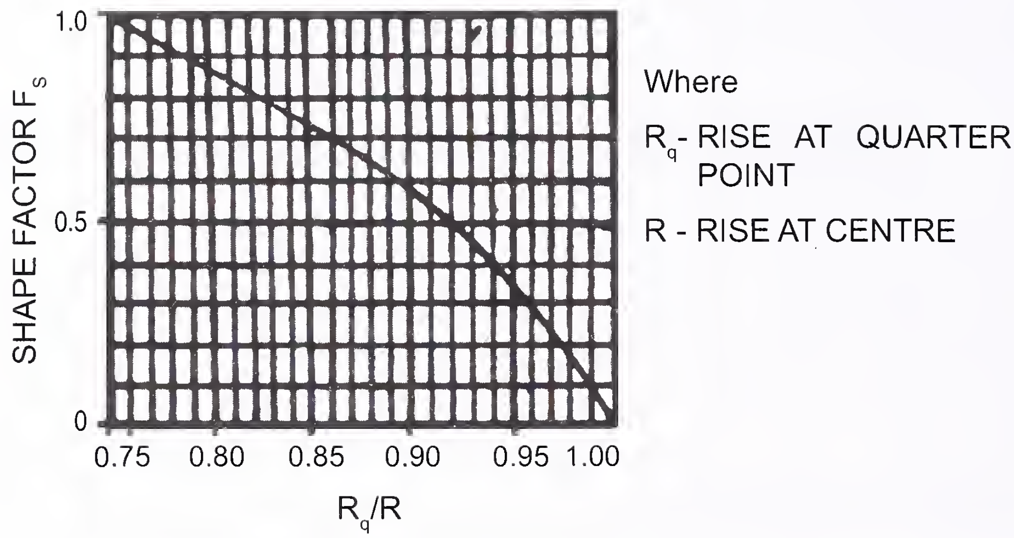


Fig. 6 Shape factors for masonry arch bridges

B. Material Factors

The material factor of an arch F_m shall be arrived at from the expression.

$$F_m = \frac{(F_r d + F_f h)}{(d + h)}$$

Where d is the arch ring-thickness h is the depth of fill F_r – the arch ring factor and F_f – the fill factor shall be as in **Tables 5 & 6**.

Table 5

Arch Ring	Ring Factor (F_r)
Granite and built-in-course masonry with large shaped voussoirs	1.50
Concrete Blocks	1.20
Lime-stone good random masonry and bricks in good condition	1.00
Masonry (of any kind) or brick work in poor condition (many voussoirs flaking or badly spalling, shearing, dilapidation is only moderate)	0.70

Table 6

Filling	Ring Factor (F_r)
Concrete slab	1.00
Lime-concrete or similar grouted material	0.90
well compacted material	0.70
Weak materials evidenced by tracking of the carriageway surface	

C. Joint Factors

The joint factor of and arch, F_j shall be arrived at from the expression.

$$F_j = F_w F_d F_{mo}$$

Where F_w the width factor F_d the depth factor and F_{mo} the mortar factor shall be as given in **Tables 7,8 and 9**.

Table 7

Width of Joint	Depth Factor (F_w)
Joints with widths upto 6 mm	1.0
Joints with widths between 6	0.8

Table 8

Depth of Joint	Depth Factor (Pd>)
Pointed joints in good condition	1.0
Unpointed joints, pointing in poor condition and joints with upto 12 mm from the edge insufficiently filled	0.9
Joints with widths from 12 mm to one tenth of the thickness of the ring insufficiently filled	0.8
Joints insuffidiently filled for more than one tenth the Thickness of the ring	At the discretion of the Engineer

Interpolation between these values is permitted, depending upon the extent and position of the joint deficiency.

Table 9

Condition of Joint	Mortar Factor (F_{mo})
Mortar in good condition	1.0
Loose or friable mortar	0.9

Rating of Bridges

D. Support Factor

Sl. No	Condition of Supports	Factor	Remarks
1.	Both abutments satisfactory	1.0	An abutment may be regarded as unsatisfactory to resist the full thrust of the arch if:
2.	One abutment unsatisfactory	0.95	
3.	Both abutments unsatisfactory	0.90	(i) The bridge is on a narrow embankment particularly if the approaches slope steeply upto the bridge.
4.	Arch carried on one abutment and one pier	0.90	(ii) The bridge is on an embarked curve.
5.	Arch carried on two piers	0.80	(iii) The abutment walls are very short and suggest little solid fill behind the arch.

E. Cracks Factor

Sl. No	Condition of Support	Factor	Remarks
1.	Longitudinal cracks within 0.6 m of the edge of the arch; if wider than 6 mm and longer than 1/10 of the span then in bridges. (a) Wider than 6 m between parapets (b) Narrower than 6 m between parapets	 1.0 0.8	Due to an outward force on the spandrel walls caused by lateral spread of the fill **Fig. 7 (a)**.
2.	Longitudinal cracks in middle third of the bridge width : (a) One small crack under 3 mm wide and shorter than 1/10 of the span b) Three or more small cracks as above c) One large crack wider than 6 mm and longer than 1/10 of the span	1.0 0.5 0.5	Due to varying amount of subsidence along the length of the abutments, large cracks are danger signs which indicate that the arch ring has broken up into narrow independent rings **Fig. 7 (b)**

3	Lateral and diagonal cracks less than 3 mm wide and shorter than 1/10 of the arch width	1.0	Lateral cracks, usually found near the quarter points, are due to permanent deformation of the arch which may be caused by partial collapse of the arch or abutment movements Diagonal crack, usually starting near the sides of the arch the springing and spreading towards the centre of the arch at the crown are probably due to subsidence at the sides of the abutment. They indicates that the bridge is in a dangerous state
4	Lateral and diagonal cracks wider than 6 mm and longer than 1/10 of the arch width Restrict the load class to 12T or the calculated class using all other applicable factor, whichever is less		
5	Crack between the arch ring and spandrel or parapet walls greater than 1/10th of the span due to spread of the GII	0.9	Due to (I) spreadeing of the fill pushing the wall outwards. Fig. 8 or (I) movement of a flexble ring away from a still fill so that the two act in dependently. This type of failure often produces cracks in the spandrel wall near the quarter points Fig. 9
6	Cracks between the arch ring and spandrel or parapet wall due to a dropped ring Reclassify from the nomogram taking the crown thickness as that of the ring alone		

F. Deformation Factors

Deformation of the Arch	Allowance to be Made	Remarks
If the deformation is limited so that the rise over the affected portion is always positive	Discard the profile factor alredy calculated and apply the span/ rise ratio of the affected portion to the whole arch	Arch ring deformation may be due to (I) Partial failure of the ring, observable in the ring itself and often accompauied by a sag in the parapet over approximately the same length. Fig. 10 or (II) movement at the abutment

G. Aburment Fault Factors

Sl. No	Condition of Support	Factor	Remarks
1	Inward movement of the abutment a) Old movement with well consolidated fill the slight hogging of the arch ring. b) Recent movement or poor fill	0.75 0.50	shown by hogging of the arch ring and parapet at the crown and possibly open cracks in the intrados between the quarter points and the springing
2	Outward spread of the abutments. If movement has been small and appears to have ceased, apply factor based on typeand condition of fill.	1.00 to 0.5	Usually cause change in the profile
3	Vertical settlement of one abutment. Apply factor varying from 0.9 for slight movement to 0.5 where the materials under each abutment are dissimilar	0.9 to 0.5	The nature of the back fill and foundations can be discovered only by probing, but this shoud be necessary only on important routes when the strength of the bridge is in doubt

General Note on Crack = Old cracks no longer operating and which probably occurred soon after the bridges was built can be ignored. Recent cracks usually show clean faces with perhaps small loose fragments of masonry. Although cracks may shear through bricks or stone, they normally follow an irregular line through the mortar. Care must be taken not to confuse such cracks with mere deficiences of the pointing material.

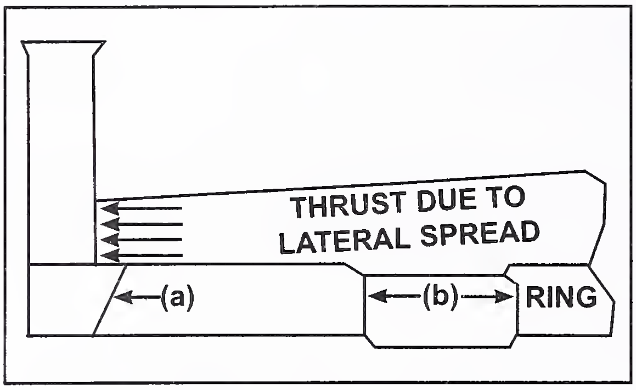


Fig. 7 Longitudinal Cracks in an Arch Ring

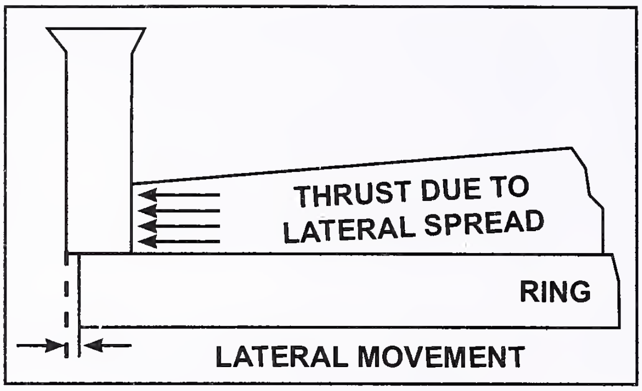


Fig. 8 Cracks Between the Arch Ring and the spandrel or Parapet Wall

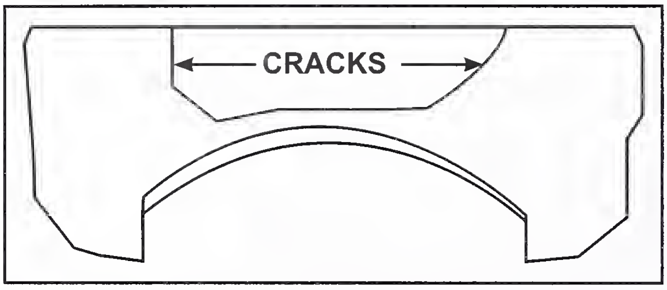


Fig. 9 Movement of the Arch Ring Away from a Stiff Fill

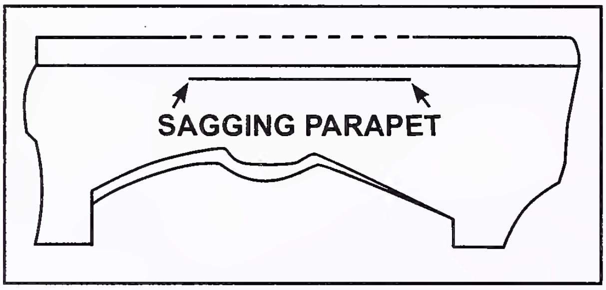


Fig. 10 Deformation of the Arch Ring

Annex 3
(Clause **9.2**)

COMPARISON BETWEEN LIVE LOAD FORCES (MAX.B.M. AT MID SPAN AND MAX SHEAR AT SUPPORT) GENERATED BY IRC LIVE LOAD AND GVW CLASS LOADS FOR SIMPLY SUPPORTED SPANS

1 GENERAL

Live load analysis has been done for simply supported spans ranging from 10 m to 75 m with an increment of 5 m in span length showing BM at mid span and shear force next to the support.

The following conditions have been taken into consideration for preparing the Tables and Graphs for bending moments and shear forces.

- For IRC Loadings, values of BM and SF are inclusive with impact factor and lane reduction factor as per IRC:6.
- For GVWs in moving traffic case, BM and SF are inclusive with impact factor, lane reduction factor as per IRC:6 and an over load factor of 1.4 Results are also furnished for GVW 49.00 T with an over load factor of 2.0.
- For GVWs in crowded/traffic jam case, BM and SF are inclusive with lane reduction factor as per IRC:6, an over load factor of 1.4 and without impact factor. Results are also furnished for GVW 49.0 T with overload factor 2.0.
- Minimum spacing between rear and front axles of two successive GVW vehicles is taken as 20 m in moving traffic case and 4.0 m in crowded/traffic jam case.
- **For spans with 1-lane carriageway (carriageway width not more than 5.3 m)**, **IRC** results are based on governing effect from 1-Lane Cl, A + UDL of 500 Kg/m^2, 1 lane Cl.70R Wheeled and 1-Lane Cl. 70R Tracked. Class A 1-Lane loading occupies 2.3 m width in a carriageway of 5.3 m and includes an UDL of 500 Kg/m^2 on remaining width of carriageway.
- **For spans with 2-lane carriageway (carriageway width between**

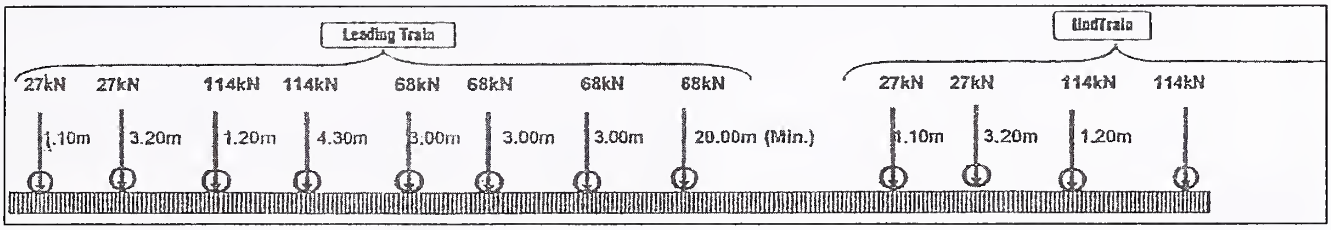
5.3 m to 9.6 m), IRC results are based on governing effect from 2-Lanes Cl. A, 1-Lane Cl. 70R Wheeled and 1-Lane Cl. 70R Tracked.

- **For spans with 3-lane carriageway (carriageway width between 9.6 m to 13.1 m), IRC** results are based on governing effect from 3-Lanes Cl, A, (1-Lane Cl. 70R Wheeled + Cl.A 1-Lane) and (1-Lane Cl. 70R Tracked + Cl. A 1 – Lane).
- **For spans with 4-lane carriageway (carriageway width between 13.1 m to 19.6 m), IRC** results are based on governing effect from 4-Lanes Cl. A, (1-Lane Cl. 70R Wheeled + Cl. A 2-Lane), (1-Lane Cl. 70R Tracked + Cl. A2- Lane), 2-Lanes Cl, 70R Wheeled and 2-Lanes Cl. 70R Tracked.
- The disposition of the vehicles in longitudinal direction in shown in **Fig. 11 and 12.**

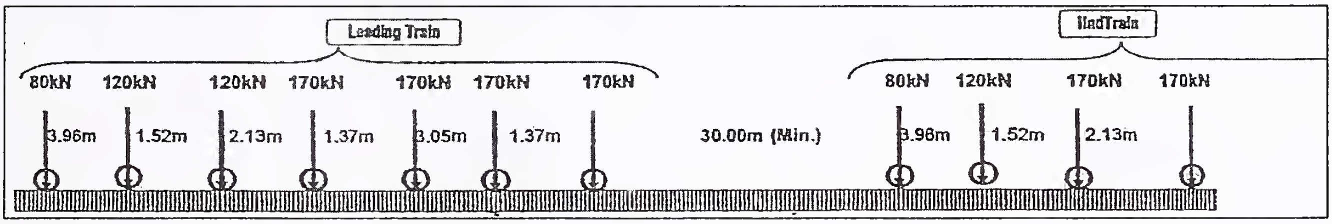
Overweight consignment (OW/OD vehicles) produces very severe effects than any other GVW class load, as well as the IRC loading, and as such this type of vehicle, needs to be analysed under special load combinations as discussed in Section 9. These are not covered in the said comparison.

As the impact factors for concrete bridges (reinforced and prestressed) are different from the steel bridges. Accordingly two sets of tables and curves have been prepared as indicated in the tables giving the MB, SF for varying bridge spans and lane widths.

(1) IRC Class A Train



(2) IRC Class 70R Wheeled Train



(3) IRC Class 70R Tracked

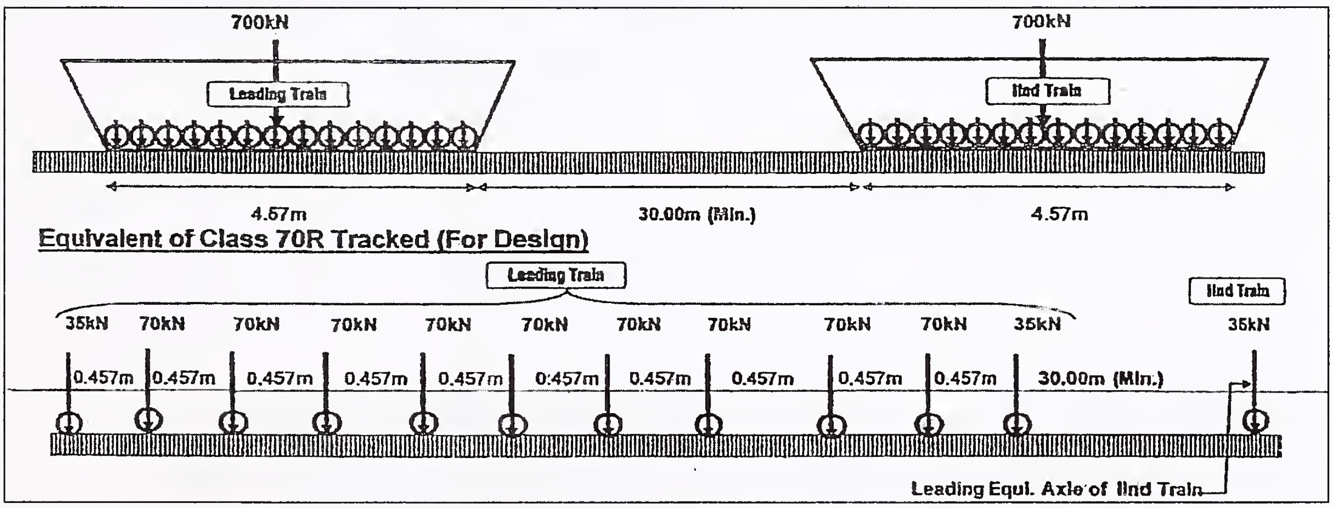
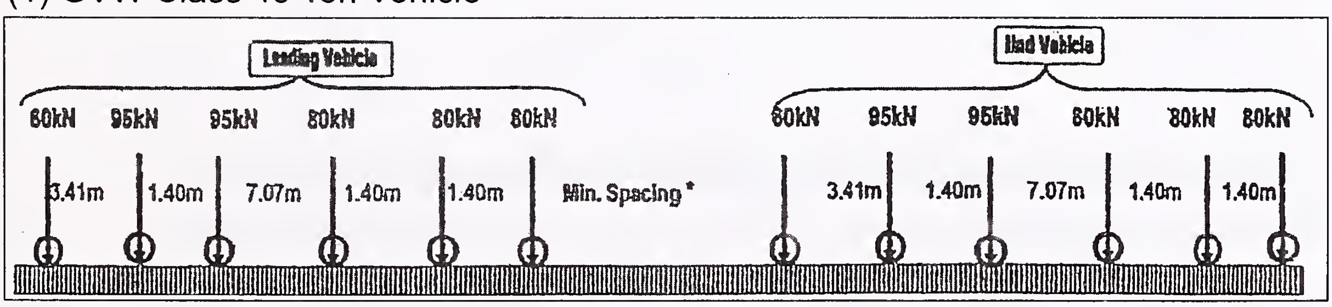


Fig. 11 Type of IRC Live Loads

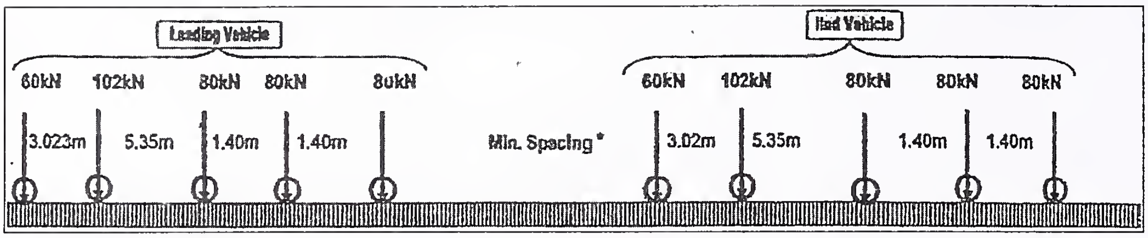
(1) GVW Class 49 Ton Vehicle



Minimum spacing between rear and front axles of two successive vehicles

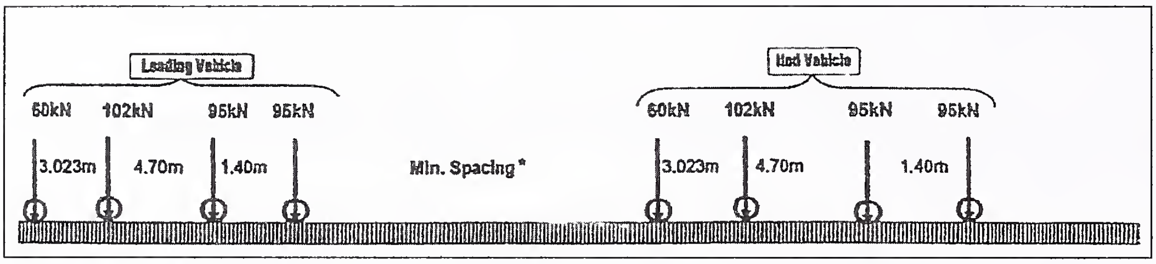
For moving traffic condition	=	20.0 m (with impact)
For crowded/Traffic Jam condition	=	4.0 m (without impact)

(2) GVW Class 40.2 Ton Vehicle



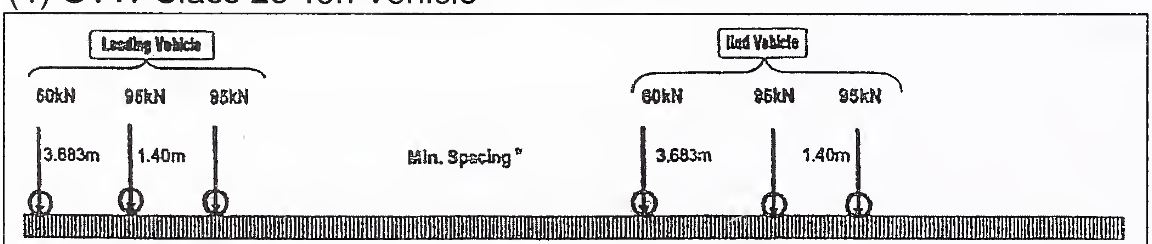
Minimum spacing between rear and front axles of two successive vehicles
For moving traffic condition = 20.0 m (with impact)
For crowded/Traffic Jam condition = 4.0 m (without impact)

(3) GVW Class 35.2 Ton Vehicle



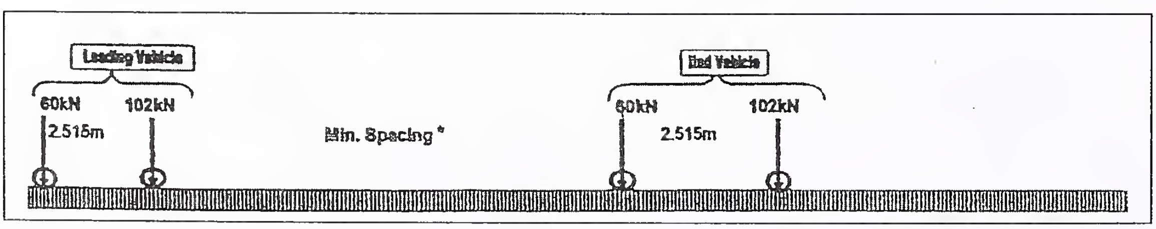
Minimum spacing between rear and front axles of two successive vehicles
For moving traffic condition = 20.0 m (with impact)
For crowded/Traffic Jam condition = 4.0 m (without impact)

(4) GVW Class 25 Ton Vehicle



Minimum spacing between rear and front axles of two successive vehicles
For moving traffic condition = 20.0 m (with impact)
For crowded/Traffic Jam condition = 4.0 m (without impact)

(5) GVW Class 16.2 Ton Vehicle



Minimum spacing between rear and front axles of two successive vehicles
For moving traffic condition = 20.0 m (with impact)
For crowded/Traffic Jam condition = 4.0 m (without impact)

Fig. 12 Type of GVW Class Loads (Adopted in Comparison)

Maximum Bending Moment at Mid Span and Shear Force at Supports Due to IRC Loads and GVW Class Loads for Simply Supported Spans (from 10 m To 75 m)

Nos. of Lanes	BM for moving Traffic Condition (t-m)	BM for Crowded/ Traffic Jam Condition (tonne-m)	Shear Force for Moving Traffic Condition (tonne)	Shear Force for Crowded/Traffic Jam Condition (tonne)	Remarks	
					For Concrete Bridges	For Steel Bridges
1-Lane	IRC loading is governing for all spans from 10 m -75 m.	IRC load governing for spans ≤ 45 m. GVW 25.0 T load governing for spans ≥ 50 m.	IRC loading is governing for all spans from 10 m -75 m.	IRC load is governing for spans ≤ 40 m. GVW 25 T load is governing for spans ≥ 45m.	Tables 10 and 11. Fig. 13, 14, 15 & 16.	Tables 18 and 19. Fig. 29, 30, 31 & 32.
2-Lane	GVW 49.0T loading is governing for all spans from 10 m -75 m.	IRC load governing for spans ≤ 15 m. GVW 40.2 T load governing for span = 20 m. GVW 25.0 T load governing for spans ≥ 25 m.	GVW 40.2T load is governing for spans ≤ 15 m. GVW 49.0 T load is governing for spans ≥ 20 m.	IRC load is governing for spans = 10 m. GVW 49 T load is governing for span = 15 m. GVW 40.2 T load is governing for span = 20 m. GVW 25.0T load is governing for spans ≥ 25 m.	Tables 12 and 13. Figs. 17, 18, 19 and 20.	Tables 20 and 21. Figs. 33, 34, 35 and 36.
3-Lane	GVW 40.2T load is governing for spans ≤ 20 m. GVW 49.0 T load governing for spans ≥ 25 m.	IRC load governing for spans ≤ 15 m. GVW 40.2 T load governing for span = 20 m. GVW 25.0T load governing for spans ≥ 25 m.	GVW 40.2T load is governing for spans ≤ 15 m. GVW 49.0 T load is governing for spans ≥ 20 m.	IRC load is governing for span = 10 m. GVW 49T load is governing for spans = 15 m. GVW 40.2 T load is governing for span = 20 m. GVW 25.0T load is governing for spans ≥ 25 m.	Table 14 and 15. Figs. 21,22, 23 and 24.	Tables 22 and 23. Figs. 37, 38, 39 and 40.
4-Lane	GVW 40.2T load is governing for spans ≤ 20 m. GVW 49.0 T load governing for spans ≥ 25 m.	IRC load governing for spans ≤ 15 m. GVW 40.2T load governing for span = 20 m. GVW 25.0 T load governing for spans ≥ 25m.	GVW 40.2 T load is governing for spans ≤ 15 m. GVW 49.0 T load is governing for spans ≥ 20 m.	IRC load governing for span =10 m. GVW 49.0T load governing for spans between 15 m to 25 m. GVW 25 T load governing for spans ≥ 30 m.	Tables 16 and 17 Figs. 25, 26, 27 and 28.	Tables 24 and 25. Figs. 41, 42, 43 and 44.

Table 10 Bending Moment for Simply Supported Span at 0.5 L Sec. for 1-Lane Carriageway: (for Concrete Superstructure)

For Carriageway width not More Than 5.3 m — Values are Inclusive with Impact and Lane Reduction Factor and a Over Load Factor of 1.4 for GVWs

Span Length (m)	B.M. in (t-m) For Moving Traffic Condition								B.M. in (t-m) For Crowded/Traffic Jam Condition							
	Due to Governing IRC Load	Due to GVW 16.2 T	Due to GVW 25.0 T	Due to GVW 35.2 T	Due to GVW 40.2 T	Due to GVW 49.0 T	Absolute Governing moment	Due to GVW 49 Ton With OLF=2	Due to Governing IRC Load	Due to GVW 16.2 T	Due to GVW 25.0 T	Due to GVW 35.2 T	Due to GVW 40.2 T	Due to GVW 49.0 T	Absolute Governing Moment	Due to GVW 49 Ton With OLF=2
10.0	149	58	78	74	85	85	149	122	149	50	63	61	68	68	149	98
15.0	266	90	129	134	143	134	266	191	266	103	115	119	124	119	266	170
20.0	396	121	176	201	216	204	396	292	396	182	197	202	207	198	396	282
25.0	530	150	222	266	292	294	530	420	530	275	315	297	306	308	530	440
30.0	661	179	267	331	366	386	661	551	661	399	447	432	418	434	661	621
35.0	791	209	312	395	439	476	791	679	791	540	594	590	581	568	791	811
40.0	920	237	357	458	511	565	920	806	920	699	779	766	764	740	920	1057
45.0	1048	278	407	526	584	653	1048	933	1048	889	990	965	959	951	1048	1359
50.0	1184	358	475	609	672	751	1184	1073	1184	1093	1215	1179	1180	1169	1215	1669
55.0	1320	451	594	707	780	872	1320	1246	1320	1323	1459	1426	1420	1403	1459	2004
60.0	1509	543	731	835	892	1013	1509	1448	1509	1574	1747	1704	1671	1669	1747	2385
65.0	1734	636	874	1001	1046	1154	1734	1649	1734	1839	2049	1998	1971	1962	2049	2803
70.0	1980	728	1017	1180	1242	1320	1980	1886	1980	2139	2366	2315	2294	2263	2366	3233
75.0	2247	821	1160	1373	1440	1539	2247	2199	2247	2455	2715	2649	2628	2583	2715	3690

For One Lane Carriageway ** Class A 1-Lane loading occupies 2.3 m width in a carriageway of 5.3 m and also includes an UDL of 500 Kg/m^2 on remaining width

Table 11 Shear Force for Simply Supported Span at 0.0 L Sec. for IRC 1-Lane Carriageway: (for Concrete Superstructure)

For Carriageway Width Not More Than 5.3 m

Values are Inclusive with Impact and Lane Reduction Factor and a Over Load Factor of 1.4 for GVWs

Span Length (m)	S.F. in (Tonne) for Moving Traffic Condition								S.F. in (Tonne) For Crowded/Traffic Jam Condition							
	Due to Governing IRC Load	Due to GVW 16.2 T	Due to GVW 25.0 T	Due to GVW 35.2 T	Due to GVW 40.2 T	Due to GVW 49.0 T	Absolute Governing Shear For	Due to GVW 49 Ton With OLF=2	Due to Governing IRC Load	Due to GVW 16.2 T	Due to GVW 25.0 T	Due to GVW 35.2 T	Due to GVW 40.2 T	Due to GVW 49.2 T	Absolute Governing Shear For	Due to GVW 49 Ton With OLF=2
10.0	64	26	36	39	39	36	64	52	64	26	30	31	32	32	64	45
15.0	80	26	38	45	48	47	80	67	80	35	41	39	40	42	80	60
20.0	87	25	37	47	51	54	87	77	87	43	49	49	50	50	87	71
25.0	91	27	37	48	53	58	91	83	91	52	59	58	59	59	91	85
30.0	93	30	41	49	54	60	93	86	93	61	69	68	68	68	93	98
35.0	95	33	46	53	57	62	95	88	95	69	78	77	77	78	95	111
40.0	96	35	49	59	62	67	96	95	96	78	88	86	87	87	96	124
45.0	97	36	52	64	68	71	97	102	97	87	98	95	96	96	98	137
50.0	103	39	54	68	73	78	103	111	103	95	107	105	105	105	107	150
55.0	112	42	59	72	78	84	112	121	112	104	117	114	114	114	117	163
60.0	120	45	63	75	81	90	120	128	120	113	127	124	123	123	127	176
65.0	128	47	67	80	85	94	128	135	128	121	136	133	133	133	136	189
70.0	134	50	70	85	90	98	134	141	134	130	146	142	142	142	146	203
75.0	140	53	73	90	96	104	140	148	140	139	155	152	151	151	155	216

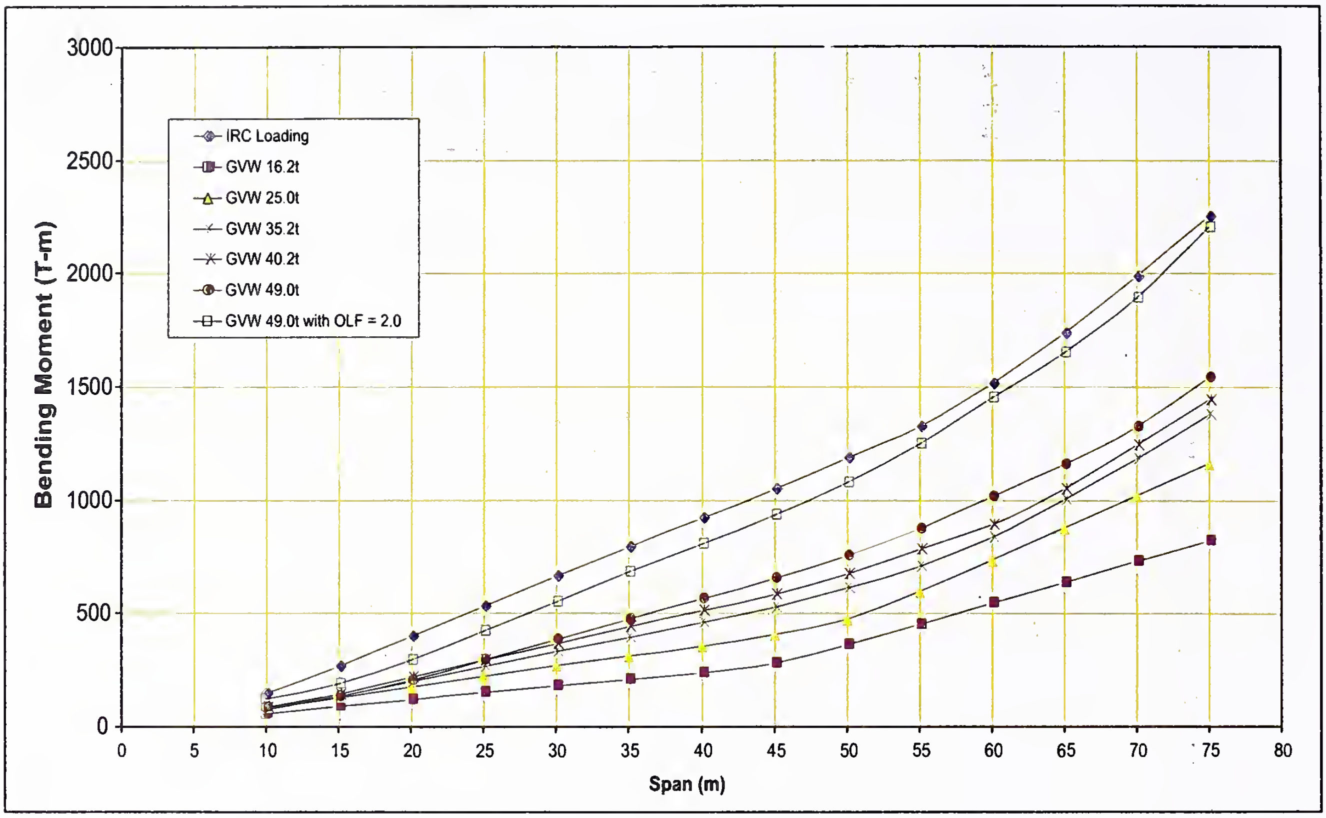


Fig. 13 Comparison of BM for IRC One Lane (For Moving Traffic Condition)

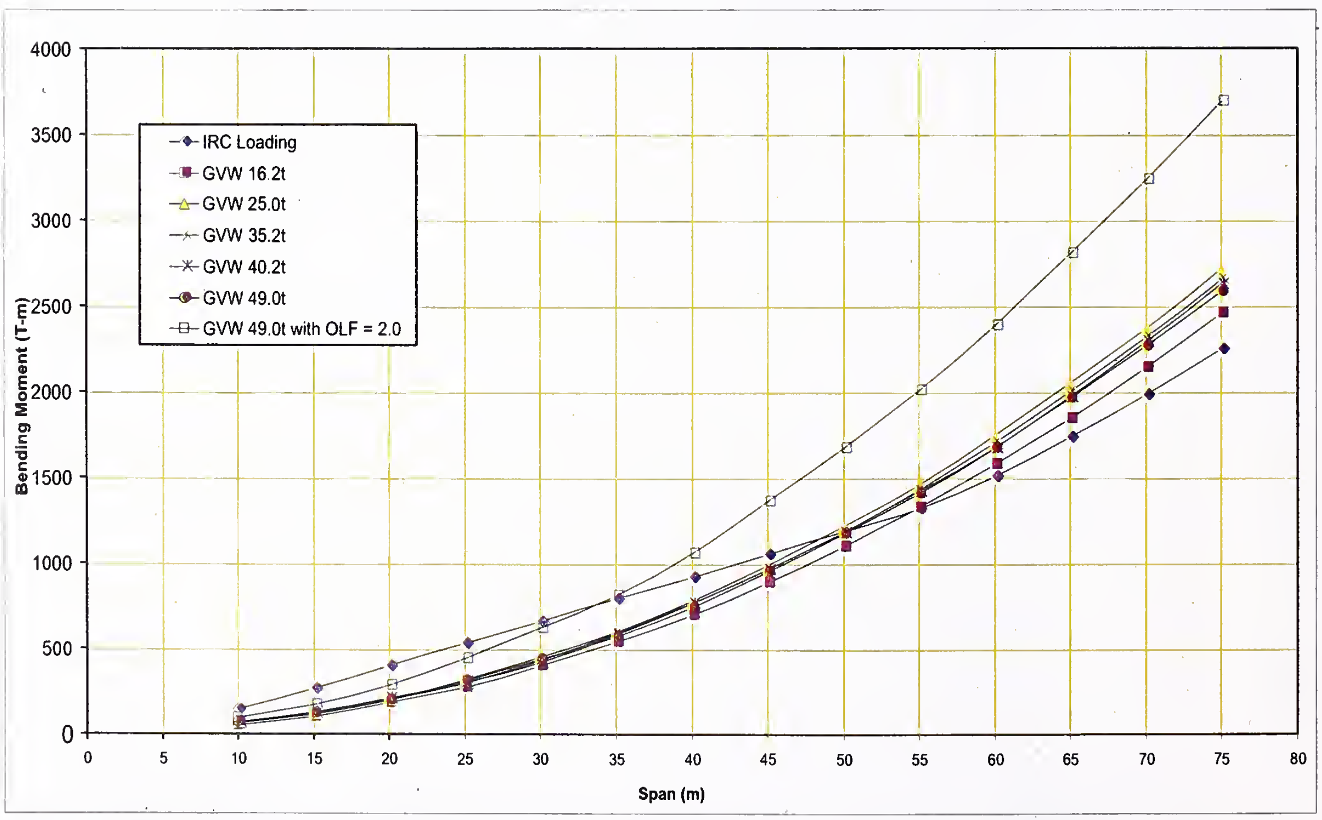


Fig. 14 Comparison of BM for IRC One Lane (For Crowded/Traffic Jam Condition)

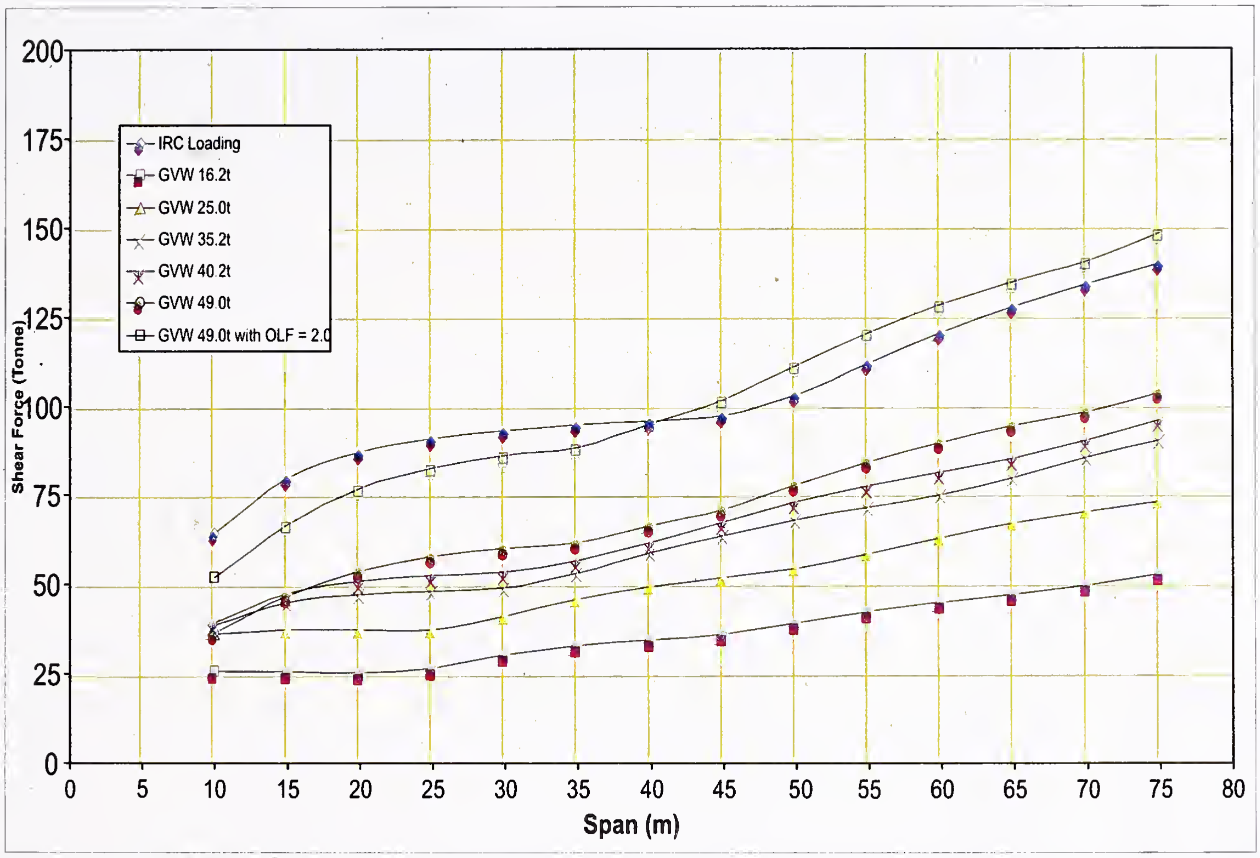


Fig. 15 Comparison of SF for IRC One Lane (For Moving Traffic Condition)

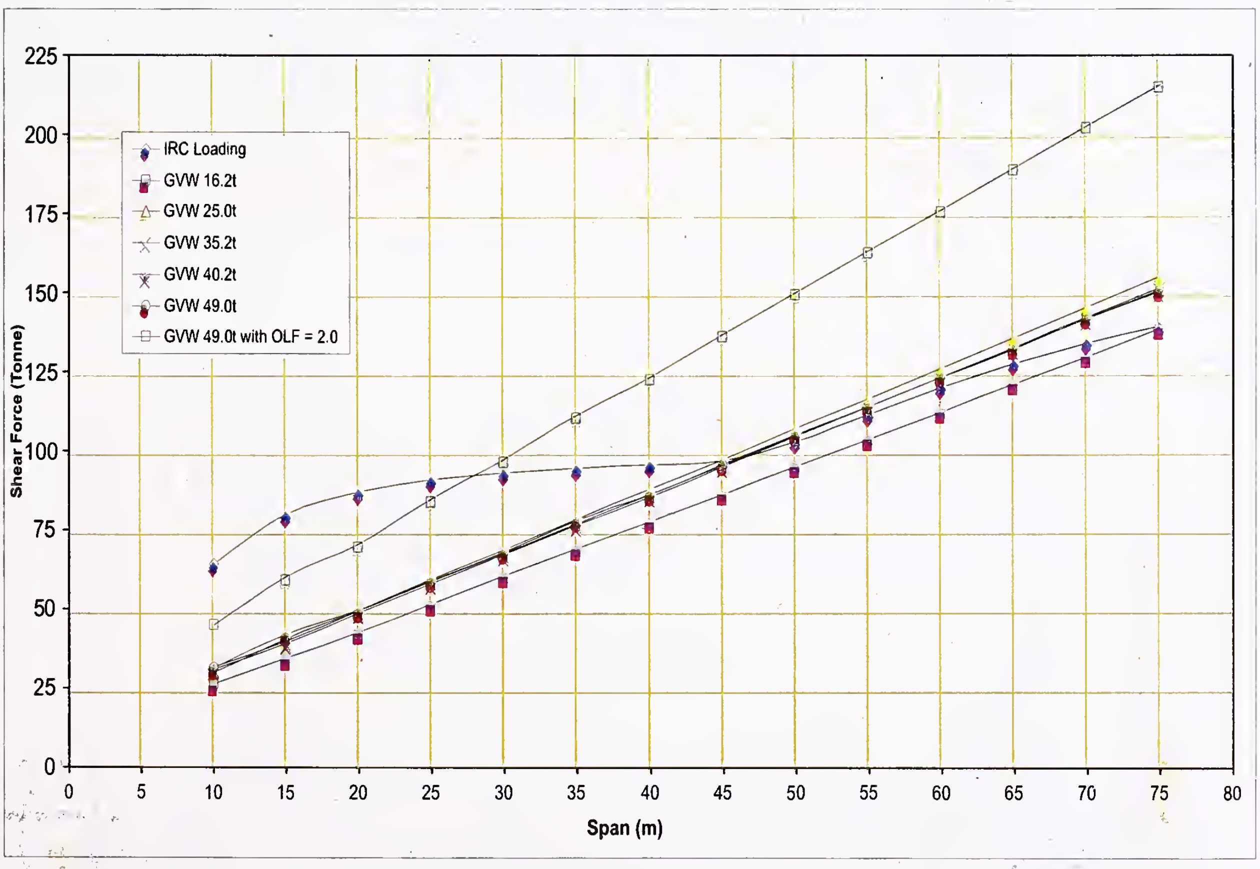


Fig. 16 Comparison of SF for IRC One Lane (For Crowded/Traffic Condition)

Table 12 Bending Moment for Simply Supported Span At 0.5 L Sec. for 2-Lanes Carriageway: (for Concrete Superstructure)

For Carriageway Width Between 5.3 m to 9.6 m   Values are Inclusive with Impact and Lane Reduction Factor and a Over Load Factor of 1.4 oor GVWs

Span Length (m)	B.M. in (t-m) for Moving Traffic Condition								B.M. in (t-m) for Crowded/Traffic Jam Condition							
	Due to Governing IRC Load	Due to GVW 16.2 T	Due to GVW 25.0 T	Due to GVW 35.2 T	Due to GVW 40.2 T	Due to GVW 49.0 T	Absolute Governing Moment	Due to GVW 49 Ton With OLF=2	Due to Governing IRC Load	Due to GVW 16.2 T	Due to GVW 25.0 T	Due to GVW 35.2 T	Due to GVW 40.2 T	Due to GVW 49.0 T	Absolute Governing Moment	Due to GVW 49 Ton With OLF=2
10.0	149	115	157	148	171	171	171	244	149	101	125	123	137	137	149	195
15.0	266	181	259	268	286	268	286	383	266	207	231	239	249	238	266	340
20.0	396	241	352	401	433	409	433	584	396	364	394	403	414	395	414	564
25.0	530	300	444	533	584	589	589	841	530	551	629	594	612	616	629	881
30.0	661	359	535	662	732	771	771	1102	661	798	894	863	837	869	894	1241
35.0	791	417	625	790	878	951	951	1359	791	1081	1187	1179	1163	1135	1187	1622
40.0	920	475	714	917	1023	1129	1129	1613	920	1398	1559	1532	1528	1480	1559	2114
45.0	1052	555	813	1053	1167	1306	1306	1866	1052	1778	1981	1929	1918	1903	1981	2718
50.0	1202	717	950	1218	1344	1502	1502	2145	1202	2187	2430	2359	2361	2337	2430	3339
55.0	1368	902	1189	1414	1559	1745	1745	2493	1368	2646	2917	2852	2839	2806	2917	4008
60.0	1548	1087	1463	1669	1783	2027	2027	2895	1548	3147	3493	3408	3343	3339	3493	4769
65.0	1743	1272	1749	2001	2092	2309	2309	3298	1743	3678	4098	3997	3942	3924	4098	5606
70.0	1962	1457	2034	2360	2483	2641	2641	3773	1962	4279	4731	4629	4588	4526	4731	6466
75.0	2199	1642	2320	2745	2881	3078	3078	4398	2199	4909	5429	5299	5256	5166	5429	7380

Table 13 Shear Force for Simply Supported Span at 0.0 L Sec. for 2-Lanes Carriageway: (for Concrete Superstructure

For Carriageway Width Between 5.3 m to 9.6 m — Values are Inclusive with Impact and Lane Reduction Factor and a Over Load Factor of 1.4 for GVWs

Span Length (M)	S.F. in (tonne) for moving traffic condition								S.F. in (tonne) for crowded / traffic jam condition							
	Due to Governing IRC Load	Due to GVW 16.2 T	Due to GVW 25.0 T	Due to GVW 35.2 T	Due to GVW 40.2 T	Due to GVW 49.0 T	Absolute Governing Shear for.	Due to GVW 49 Ton with OLF=2	Due to Governing IRC Load	Due to GVW 16.2 T	Due to GVW 25.0 T	Due to GVW 35.2 T	Due to GVW 40.2 T	Due to GVW 49.0 T	Absolute Governing Shear for.	Due to Gvw 49 Ton with OLF=2
10.0	64	51	72	78	79	73	79	104	64	53	60	62	64	63	64	90
15.0	80	52	75	90	95	94	95	134	80	69	81	78	80	85	85	121
20.0	87	51	75	94	102	108	108	154	87	86	99	98	100	99	100	142
25.0	91	53	75	96	105	116	116	165	91	104	119	116	118	119	119	170
30.0	93	61	83	98	107	120	120	172	93	121	138	135	136	137	138	195
35.0	95	66	92	107	114	124	124	177	95	138	157	154	155	156	157	223
40.0	96	69	99	118	124	133	133	190	96	156	176	172	173	173	176	247
45.0	98	72	104	127	135	142	142	203	98	173	195	191	191	192	195	274
50.0	105	79	109	136	146	156	156	222	105	191	215	210	210	210	215	301
55.0	114	85	117	143	155	169	169	241	114	208	233	228	229	229	233	327
60.0	124	90	126	150	163	180	180	256	124	225	253	247	247	247	253	353
65.0	133	95	134	160	170	189	189	270	133	243	272	266	266	265	272	379
70.0	141	100	141	171	181	197	197	281	141	260	292	285	284	284	292	406
75.0	148	106	147	181	192	207	207	296	148	278	311	304	302	302	311	431

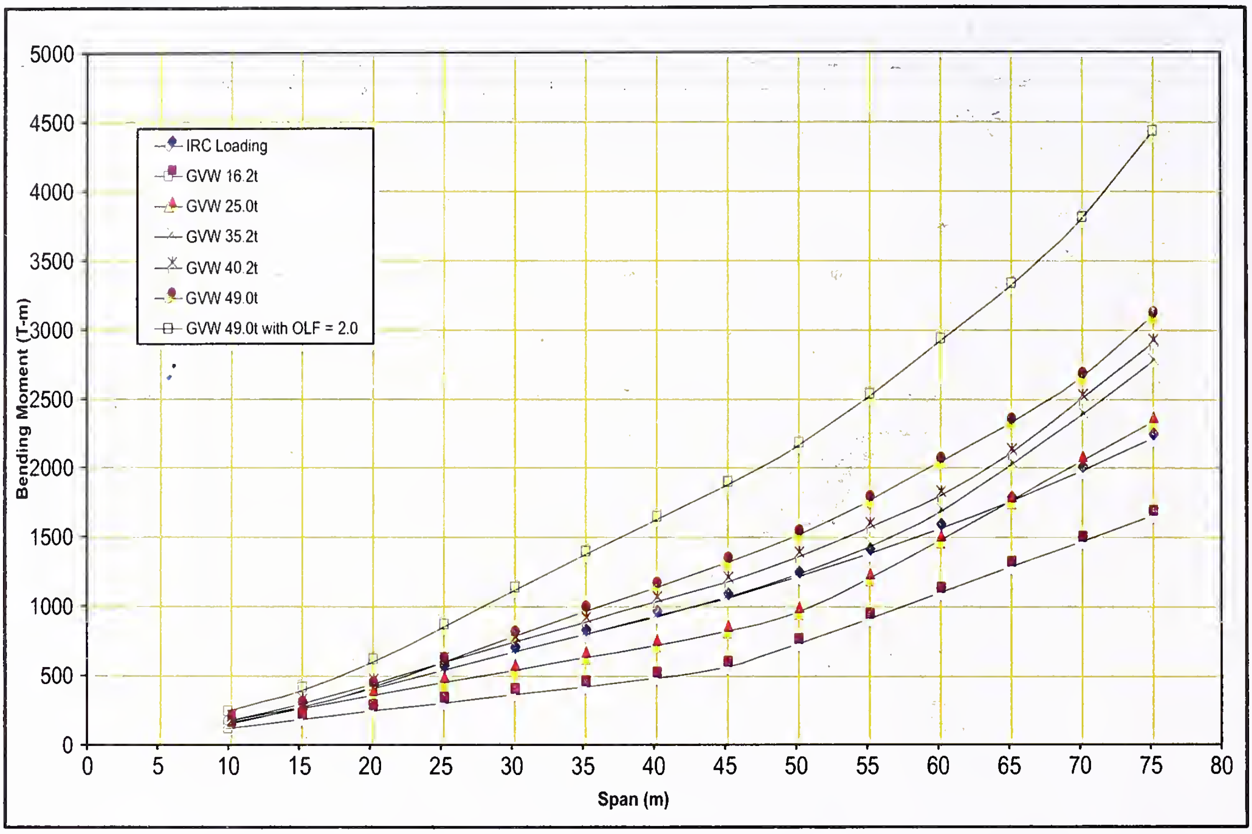


Fig. 17 Comparison of BM for IRC Two Lanes (For Moving Traffic Condition)

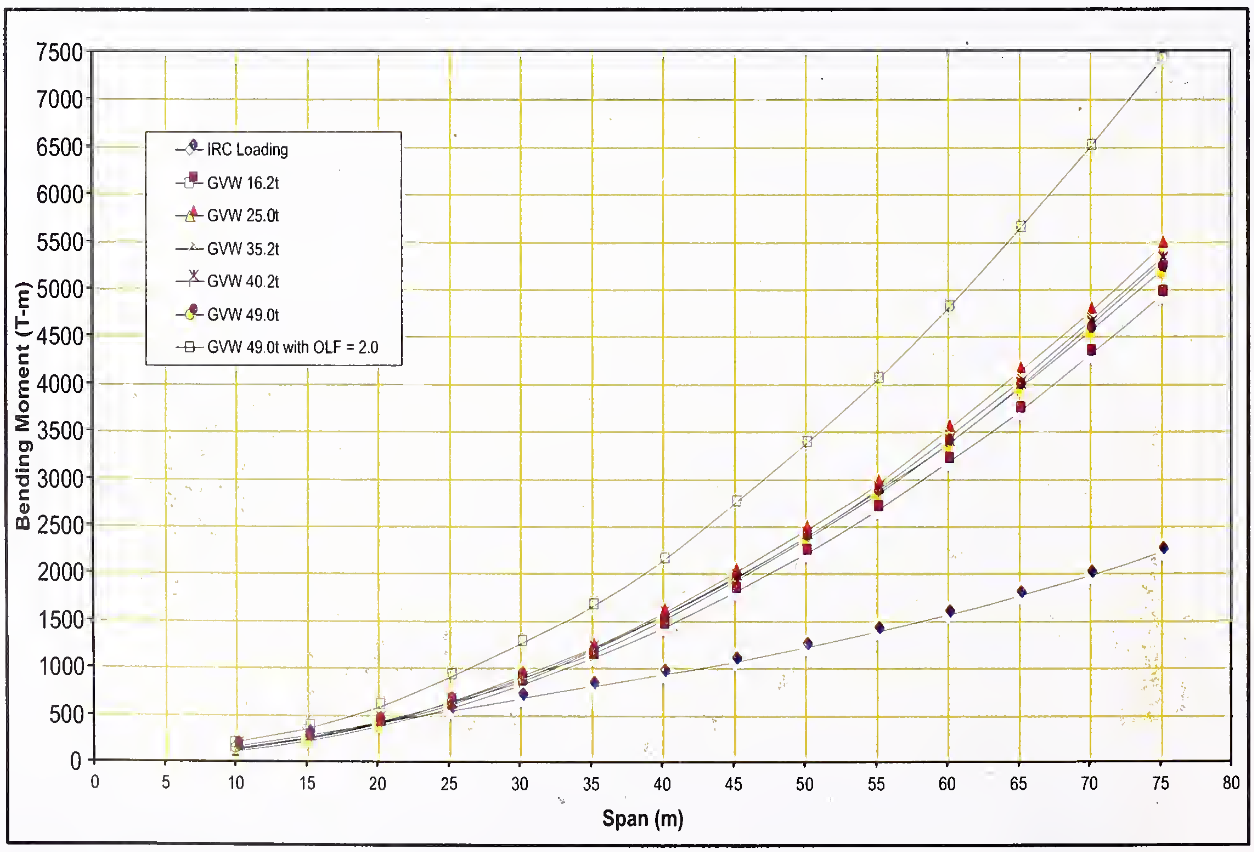


Fig. 18 Comparison of BM for IRC Two Lanes (For Crowded/Traffic Jam Condition)

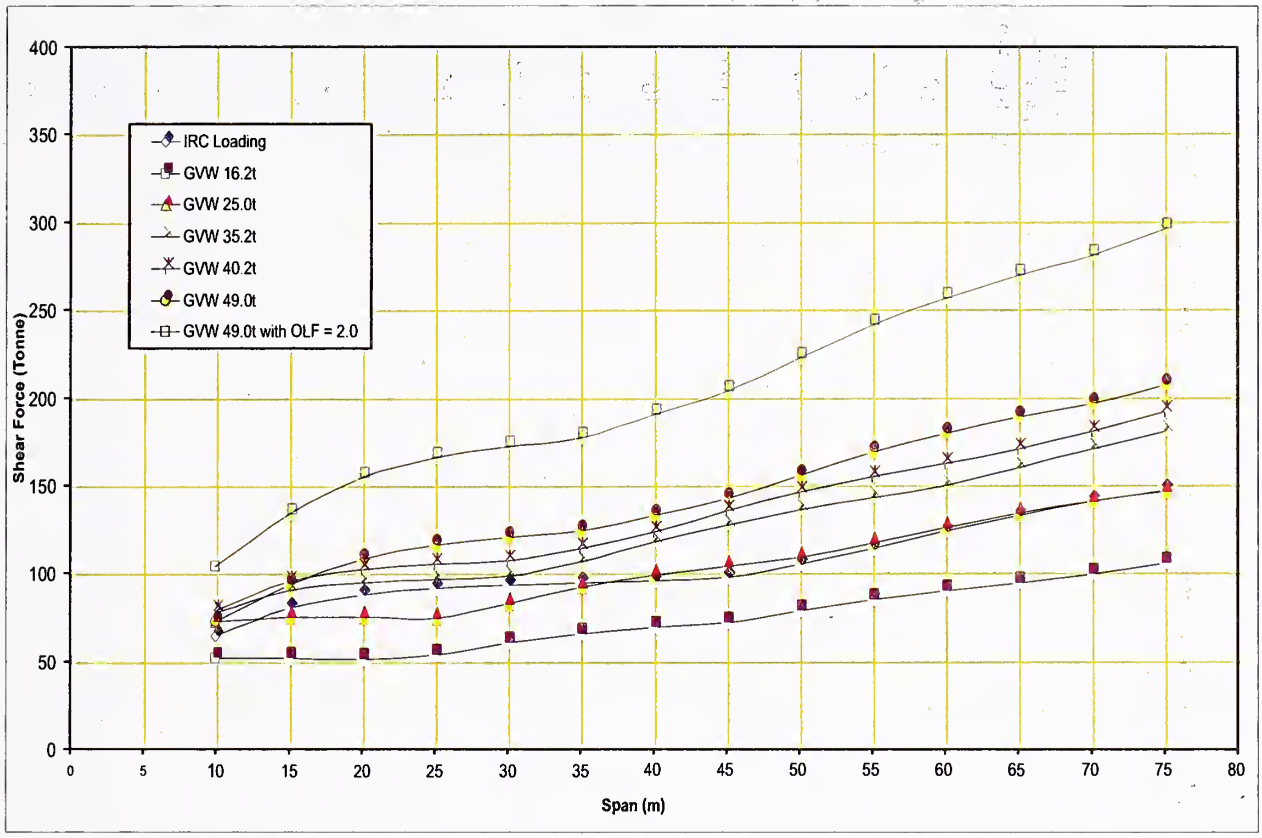


Fig. 19 Comparison of SF for IRC Two Lanes (For Moving Traffic Condition)

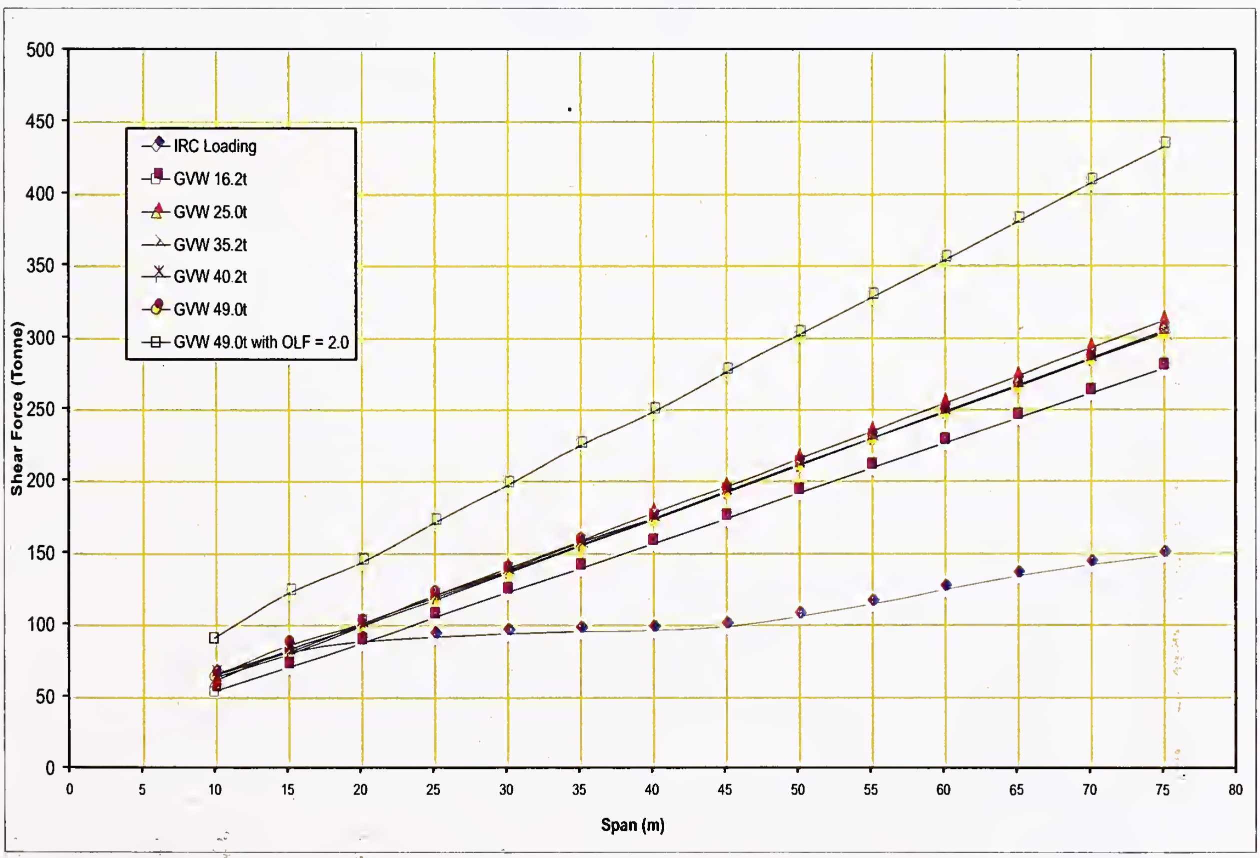


Fig. 20 Comparison of SF for IRC Two Lanes (For Crowded/Traffic Jam Condition)

Table 14 Bending Moment for Simply Supported Span at 0.5 L Sec. for 3-Lanes Carriageway: (for Concrete Sup erstructure)

For Carriageway width Between 9.6 m to 13.1 m	Values are Inclusive with Impact and Lane Reduction Factor and a Over Load Factor of 1.4 For GVWs															
Span Length (M)	B.M. in (T-M) for Moving Traffic Condition								B.M. in (T-M) for Crowded / Traffic Jam Condition							
	Due to Governing IRC Load	Due to GVW 16.2 T	Due to GVW 25.0 T	Due to GVW 35.2 T	Due to GVW 40.2 T	Due to GVW 49.0 T	Absolute Governing Moment	Due to Gvw 49 Ton With OLF=2	Due to Governing IRC load	Due to GVW 16.2 T	Due to GVW 25.0 T	Due to GVW 35.2 T	Due to GVW 40.2 T	Due to GVW 49.0 T	Absolute Governing Moment	Due to Gvw 49 Ton With OLF=2
10.0	195	156	212	200	231	231	231	329	195	136	169	166	184	184	195	264
15.0	345	244	349	362	386	362	386	517	345	279	311	322	336	322	345	460
20.0	514	326	476	542	584	552	584	788	514	491	532	545	559	533	559	762
25.0	692	406	600	719	788	795	795	1135	692	743	850	802	826	832	850	1189
30.0	874	485	722	894	988	1041	1041	1487	874	1077	1207	1165	1130	1173	1207	1676
35.0	1056	563	843	1066	1185	1284	1284	1834	1056	1459	1603	1592	1570	1533	1603	2189
40.0	1237	641	964	1238	1381	1524	1524	2178	1237	1888	2104	2069	2063	1998	2104	2854
45.0	1420	750	1098	1421	1576	1763	1763	2518	1420	2400	2674	2605	2589	2568	2674	3669
50.0	1623	968	1283	1645	1815	2027	2027	2896	1623	2952	3281	3185	3187	3155	3281	4508
55.0	1847	1217	1605	1909	2105	2356	2356	3365	1847	3572	3938	3850	3833	3788	3938	5411
60.0	2090	1467	1975	2253	2408	2736	2736	3909	2090	4249	4716	4601	4513	4507	4716	6439
65.0	2354	1717	2361	2701	2824	3117	3117	4452	2354	4965	5532	5396	5321	5298	5532	7568
70.0	2648	1967	2746	3186	3352	3565	3565	5093	2648	5776	6387	6249	6194	6110	6387	8729
75.0	2969	2217	3132	3706	3889	4156	4156	5937	2969	6628	7329	7153	7095	6974	7329	9963

Table 15 Shear Force for Simply Supported Span at 0.0l Sec. for 3-Lanes Carriageway: (for Concrete Superstructure)

For Carriageway Width Between 9.6 m to 13.1 m								Values are Inclusive with Impact and Lane Reduction Factor and a Over Load Factor of 1.4 for GVWs								
Span Length (m)	S.F. in (tonne) for Moving Traffic Condition								S.F. in (tonne) for Crowded/Traffic Jam Condition							
	Due to Governing IRC Load	Due to GVW 16.2 T	Due to GVW 25.0 T	Due to GVW 35.2 T	Due to GVW 40.2 T	Due to GVW 49.0 T	Absolute Governing Shear For.	Due to GVW 49 Ton With OLF=2	Due to Governing IRC Load	Due to GVW 16.2 T	Due to GVW 25.0 T	Due to GVW 35.2 T	Due to GVW 40.2 T	Due to GVW 49.0 T	Absolute Governing Shear For.	Due to GVW 49 Ton With OLF=2
10.0	86	69	97	105	106	98	106	140	86	71	81	84	86	85	86	122
15.0	100	70	101	122	128	126	128	180	100	93	110	106	108	114	114	163
20.0	109	68	101	127	138	146	146	208	109	116	134	132	135	134	135	191
25.0	117	72	101	130	142	156	156	223	117	140	161	157	159	160	161	229
30.0	122	82	111	132	145	163	163	232	122	164	186	182	183	184	186	264
35.0	125	89	124	144	154	167	167	239	125	187	212	208	209	211	212	301
40.0	128	93	133	159	167	180	180	257	128	210	238	232	234	234	238	334
45.0	132	97	140	172	183	192	192	274	132	234	263	258	258	259	263	370
50.0	142	106	147	184	197	210	210	300	142	257	290	283	284	284	290	406
55.0	154	115	158	193	209	228	228	325	154	281	315	308	309	309	315	441
60.0	168	122	170	203	220	242	242	346	168	304	342	334	333	333	342	476
65.0	180	128	181	216	230	255	255	364	180	327	367	359	358	358	367	512
70.0	190	134	190	231	244	266	266	379	190	351	394	384	384	383	394	548
75.0	199	143	198	244	260	280	280	400	199	375	419	410	408	407	419	582

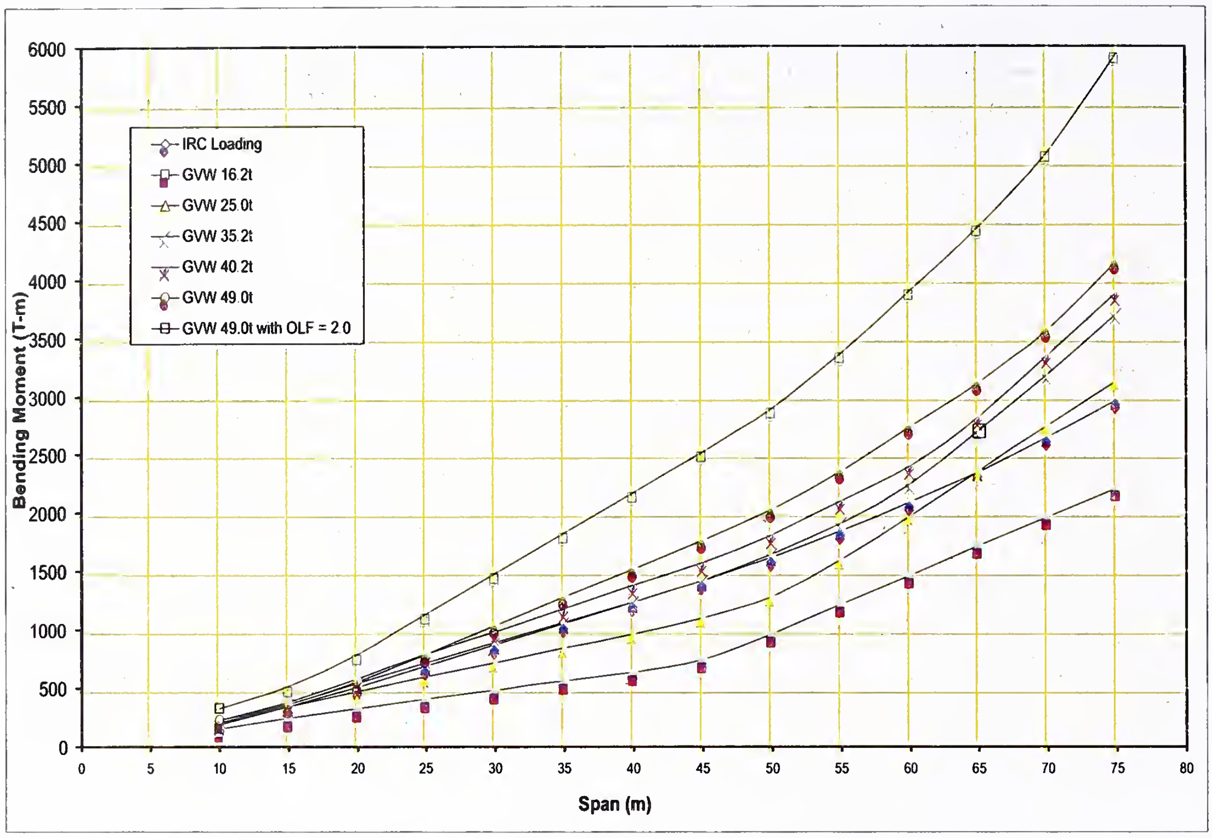


Fig. 21 Comparison of BM for IRC Three Lanes (For Moving Traffic Condition)

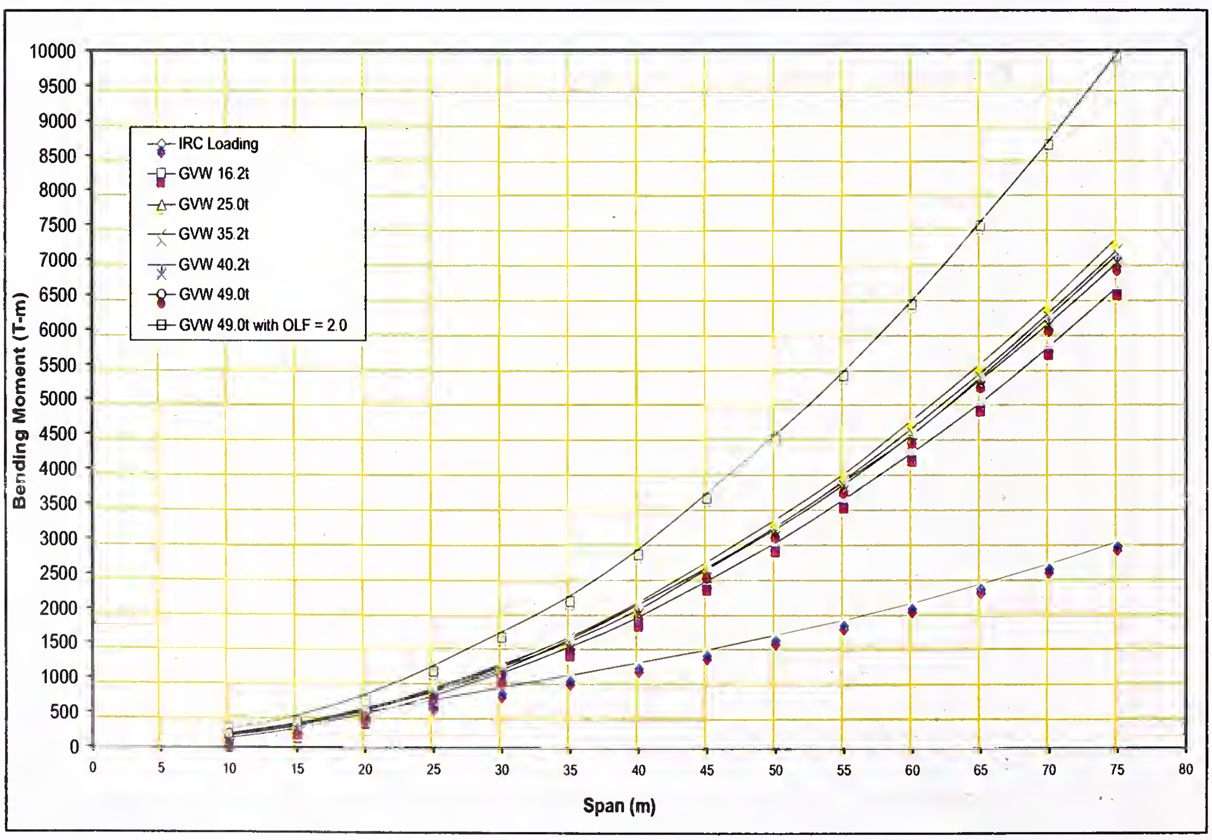


Fig. 22 Comparison of BM for IRC Three Lanes (For Crowded/Traffic Jam Condition)

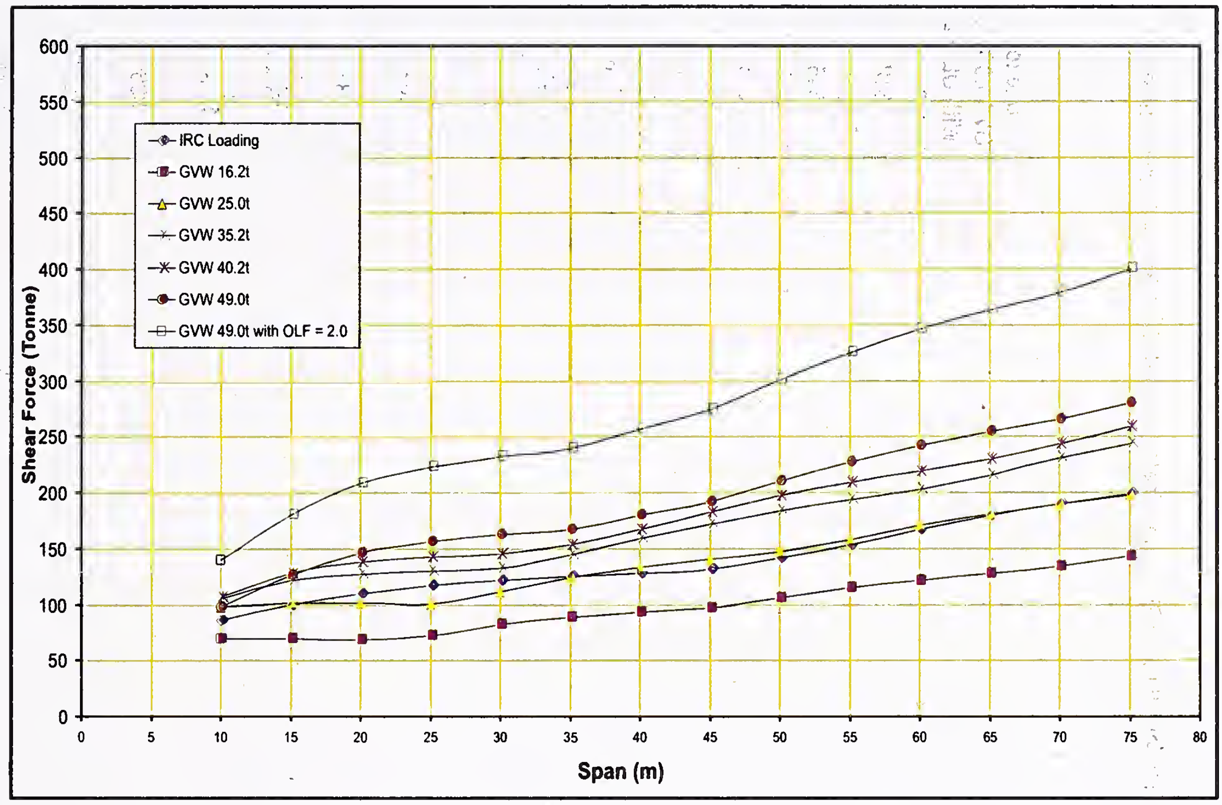


Fig. 23 Comparison of SF for IRC Three Lanes (For Moving Traffic Condition)

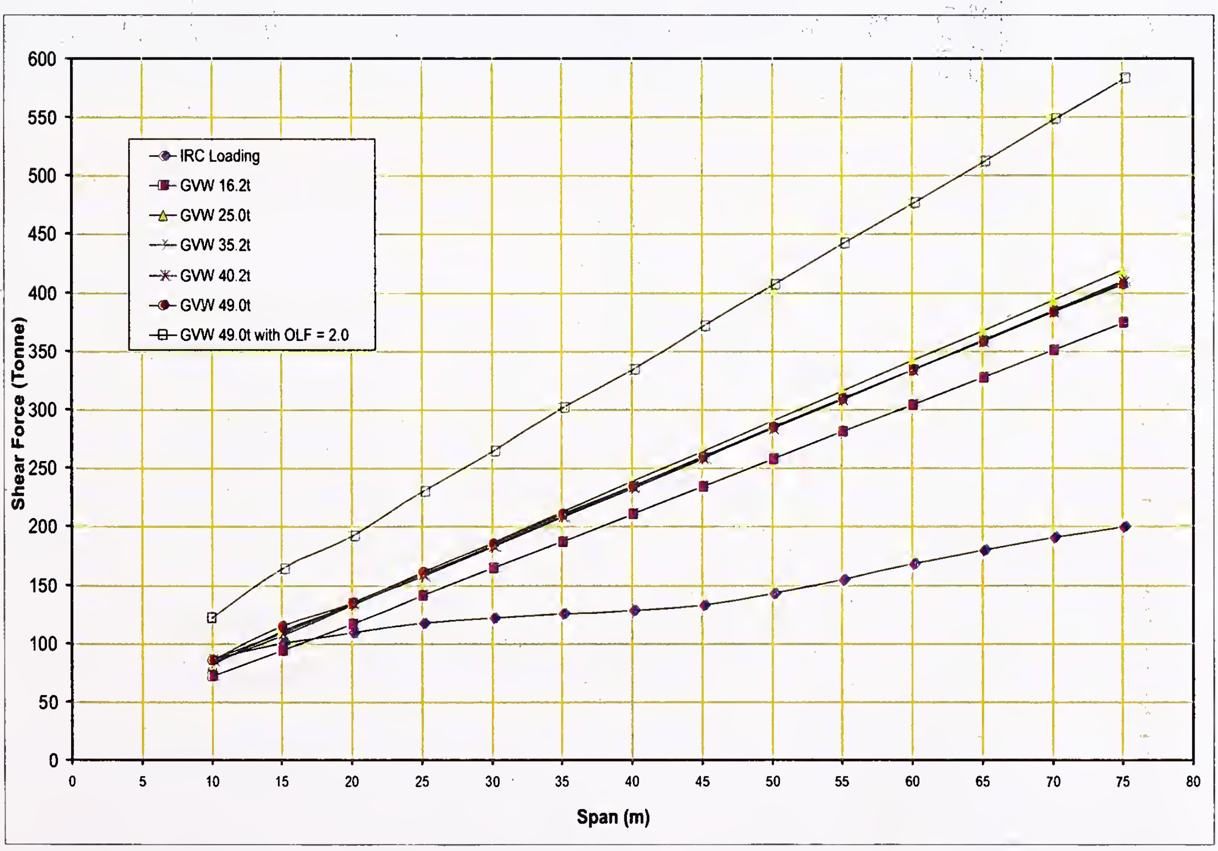


Fig. 24 Comparison of SF for IRC Three Lanes (For Crowded/Traffic Jam Condition)

Table 16 Bending Moment for Simply Supported Span at 0.5 L Sec. for 4-Lanes Carriageway: (for Concrete Superstructure

For Carriageway width Between 13.1 m to 19.6 m — Values are Inclusive with Impact and Lane Reduction Factor and a Over Load Factor of 1.4 for GVWs

Span Length (m)	B.M. in (T-M) for Moving Traffic Condition								B.M. in (t-m) for Crowded / Traffic Jam Condition							
	Due to Governing IRC Load	Due to GVW 16.2 T	Due to GVW 25.0 T	Due to GVW 35.2 T	Due to GVW 40.2 T	Due to GVW 49.0 T	Absolute Governing Moment	Due to GVW 49 Ton With OLF=2	Due to Governing IRC Load	Due to GVW 16.2 T	Due to GVW 25.0 T	Due to GVW 35.2 T	Due to GVW 40.2 T	Due to GVW 49.0 T	Absolute Governing Moment	Due to GVW 49 Ton With OLF=2
10.0	238	185	251	237	273	273	273	390	238	161	201	196	219	219	238	312
15.0	425	289	414	429	458	429	458	612	425	330	369	382	398	381	425	545
20.0	634	386	564	642	693	654	693	934	634	582	631	645	663	632	663	903
25.0	848	481	711	853	934	942	942	1345	848	881	1007	951	978	986	1007	1409
30.0	1058	574	856	1059	1171	1234	1234	1763	1058	1277	1430	1381	1339	1390	1430	1986
35.0	1266	667	1000	1264	1405	1522	1522	2174	1266	1729	1900	1887	1860	1816	1900	2595
40.0	1472	760	1143	1467	1637	1807	1807	2581	1472	2237	2494	2452	2445	2368	2494	3383
45.0	1683	888	1301	1684	1867	2089	2089	2985	1683	2845	3169	3087	3069	3044	3169	4349
50.0	1924	1147	1520	1949	2151	2402	2402	3432	1924	3499	3888	3774	3777	3740	3888	5342
55.0	2189	1443	1902	2263	2495	2792	2792	3989	2189	4234	4668	4563	4543	4489	4668	6413
60.0	2477	1739	2341	2671	2853	3243	3243	4633	2477	5036	5589	5453	5348	5342	5589	7631
65.0	2789	2035	2798	3202	3347	3694	3694	5277	2789	5885	6557	6395	6307	6279	6557	8969
70.0	3139	2331	3255	3776	3973	4225	4225	6036	3139	6846	7570	7407	7342	7242	7570	10345
75.0	3519	2627	3712	4392	4609	4925	4925	7036	3519	7855	8686	8478	8409	8266	8686	11808

Table 17 Shear Force for Simply Supported Span at 0.0 L Sec. for 4-Lanes Carriageway: (for Concrete Superstructure)

For Carriageway width Between 13.1 m to 19.6 m

Values are Inclusive with Impact and Lane Reduction Factor and a Over Load Factor of 1.4 for GVWs

Span Length (m)	S.F. in (tonne) for Moving Traffic Condition								S.F. in (tonne) for Crowded / Traffic Jam Condition							
	Due to Governing IRC Load	Due to GVW 16.2 T	Due to GVW 25.0 T	Due to GVW 35.2 T	Due to GVW 40.2 T	Due to GVW 49.0 T	Absolute Governing Shear For	Due to GVW 49 Ton With OLF=2	Due to Governing IRC Load	Due to GVW 16.2 T	Due to GVW 25.0 T	Due to GVW 35.2 T	Due to GVW 40.2 T	Due to GVW 49.0 T	Absolute Governing Shear For	Due to GVW 49 Ton With OLF=2
10.0	103	82	115	124	126	116	126	166	103	84	96	99	102	101	103	144
15.0	128	83	120	144	152	150	152	214	128	111	130	125	128	135	135	193
20.0	140	81	120	151	163	173	173	247	140	138	158	157	159	159	159	227
25.0	146	85	119	154	169	185	185	265	146	166	190	186	188	190	190	272
30.0	149	97	132	157	172	193	193	275	149	194	220	216	217	219	220	312
35.0	152	105	147	170	182	198	198	283	152	221	251	246	247	250	251	357
40.0	153	111	158	188	198	213	213	304	153	249	282	275	277	277	282	396
45.0	156	115	166	204	216	228	228	325	156	277	312	306	306	307	312	439
50.0	168	126	174	218	234	249	249	356	168	305	344	336	336	337	344	481
55.0	182	136	187	229	248	270	270	386	182	333	374	365	366	366	374	523
60.0	199	144	202	240	260	287	287	410	199	360	405	396	395	395	405	564
65.0	213	151	215	256	273	302	302	431	213	388	435	425	425	424	435	606
70.0	225	159	225	273	289	315	315	450	225	416	467	456	455	454	467	649
75.0	236	170	235	289	308	332	332	474	236	444	497	486	484	483	497	690

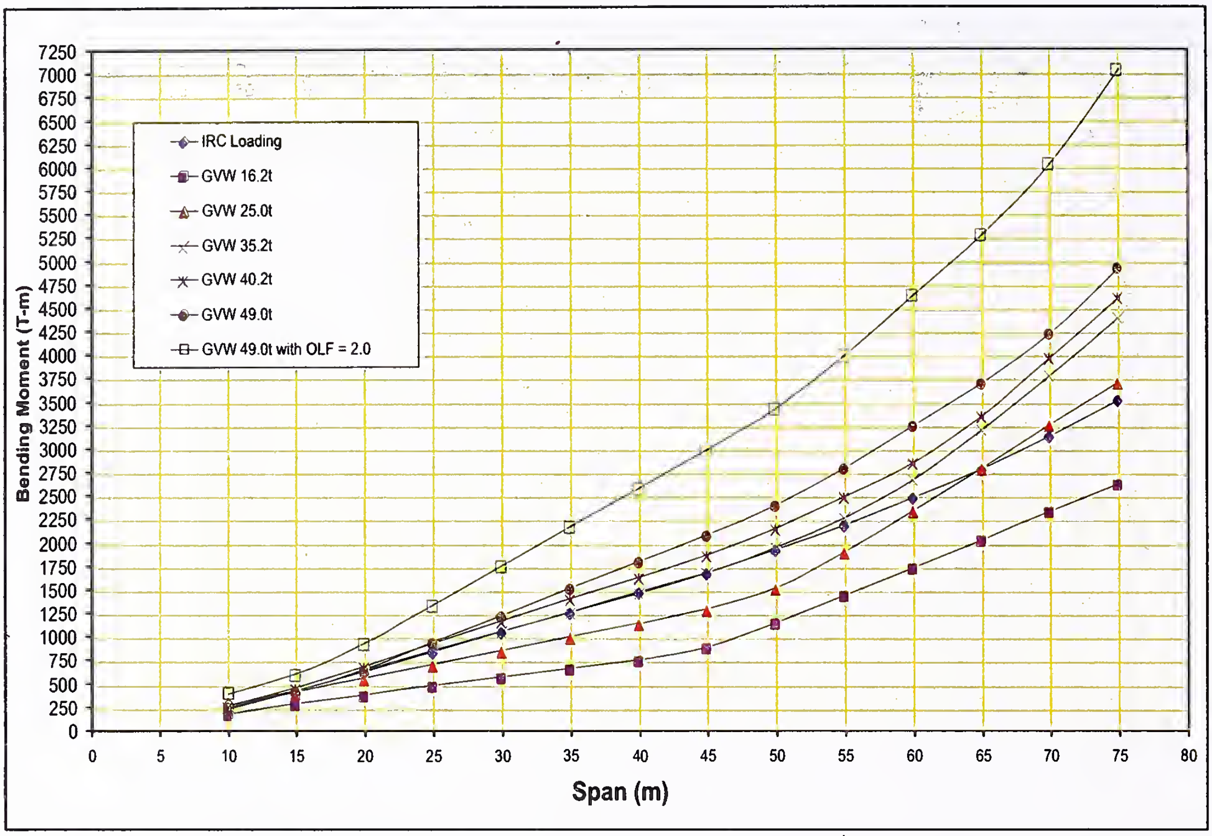


Fig. 25 Comparison of BM for IRC Four Lanes (For Moving Traffic Condition)

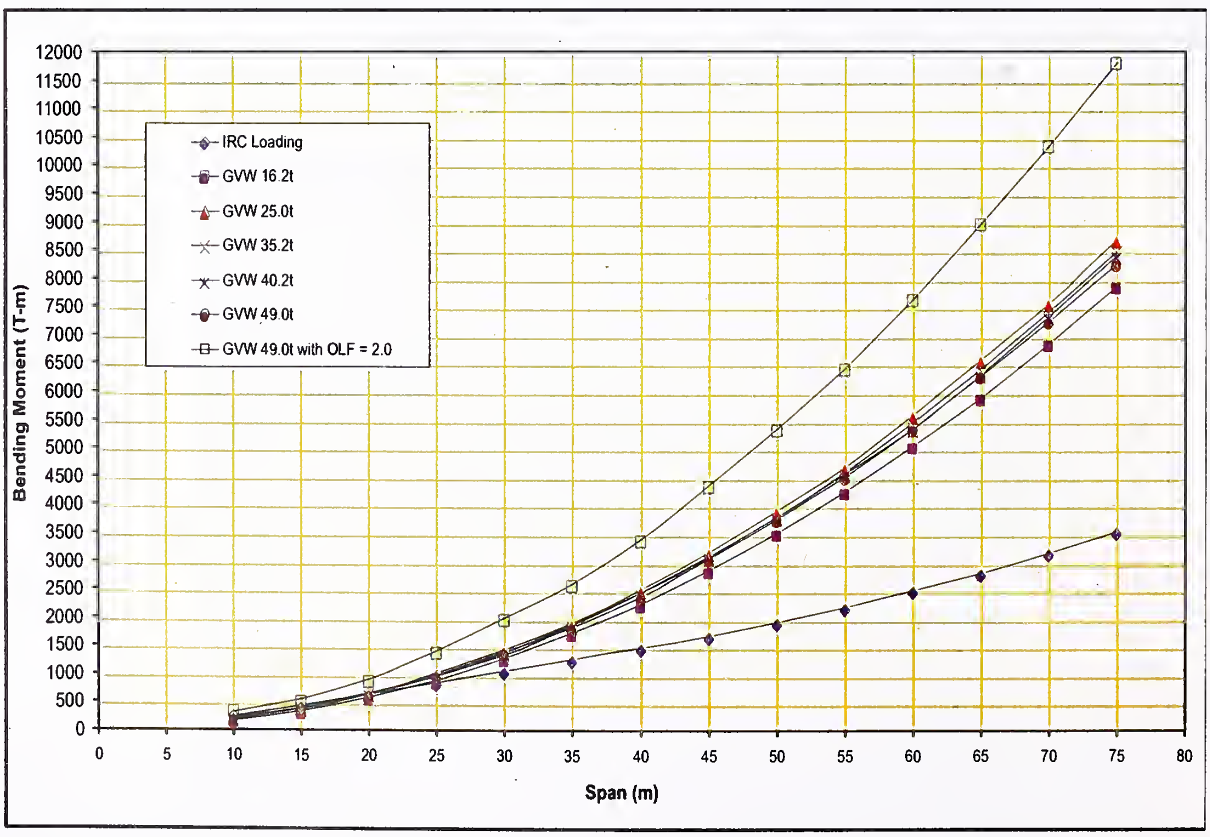


Fig. 26 Comparison of BM for IRC Four Lanes (For Crowded/Traffic Jam Condition)

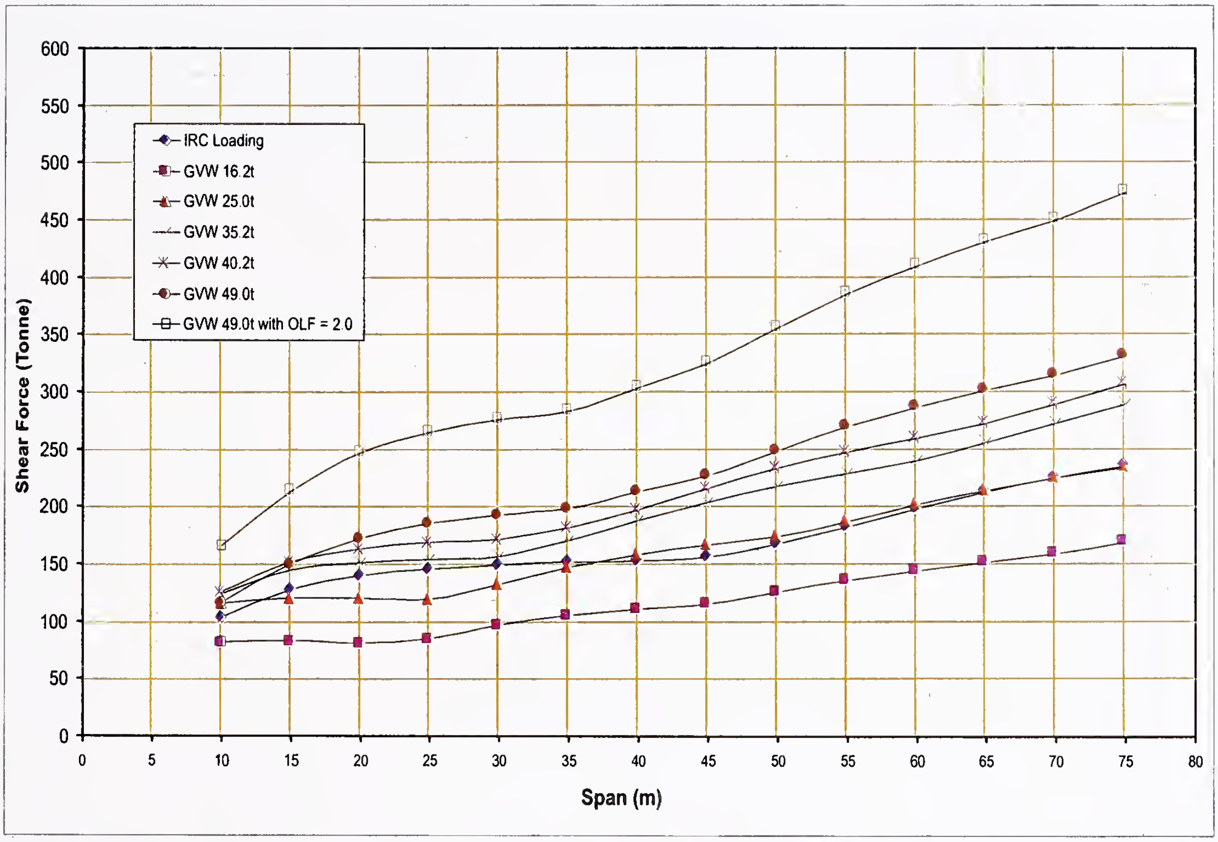


Fig. 27 Comparison of SF for IRC Four Lanes (For Moving Traffic Condition)

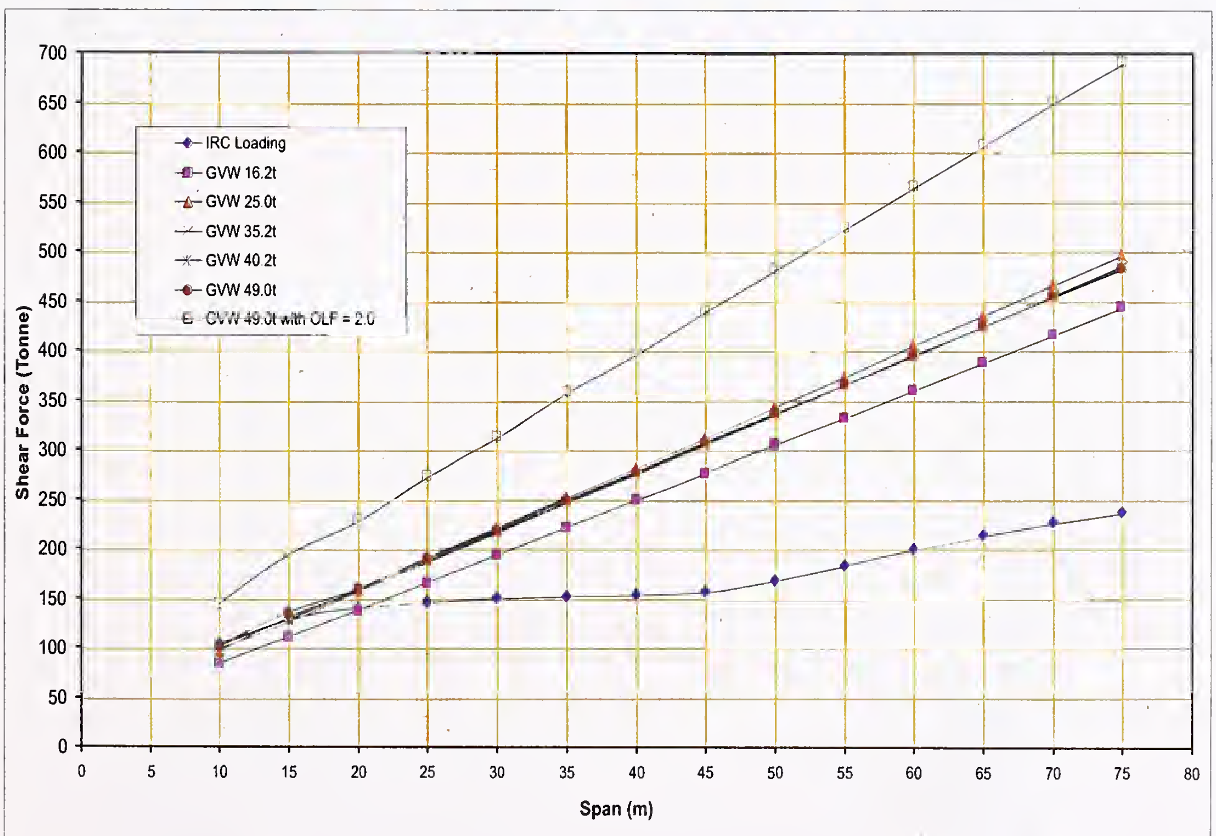


Fig. 28 Comparison of SF for IRC Four Lanes (For Crowded/Traffic Jam Condition)

Table 18 Bending Moment for Simply Supported Span At 0.5 L Sec for 1-Lane Carriageway: (for Steel Superstructure)

For Carriageway Width not more than 5.3 m	Values are Inclusive with Impact and Lane Reduction Factor and a Over Load Factor of 1. 4 for GVWs															
Span Length (m)	B.M. in (t-m) For Moving Traffic Condition								B.M. in (t-m) For Crowded / Traffic Jam Condition							
	Due to Governing IRC Load	Due to GVW 16.2T	Due to GVW 25.0T	Due to GVW 35.2T	Due to GVW 40.2T	Due to GVW 49.0T	Absolute Governing Moment	Due to GVW 49 Ton With OLF=2	Due to Governing IRC Load	Due to GVW 16.2T	Due to GVW 25.0T	Due to GVW 35.2T	Due to GVW 40.2T	Due to GVW 49.0T	Absolute Governing Moment	Due to GVW 49 Ton With OLF=2
10.0	149	58	78	74	85	85	149	122	149	50	63	61	68	68	149	98
15.0	273	93	133	138	147	138	273	197	273	103	115	119	124	119	273	170
20.0	422	129	188	214	231	218	422	311	422	182	197	202	207	198	422	282
25.0	571	162	239	287	314	317	571	453	571	275	315	297	306	308	571	440
30.0	709	192	287	355	392	413	709	591	709	399	447	432	418	434	709	621
35.0	845	223	334	422	469	508	845	725	845	540	594	590	581	568	845	811
40.0	978	253	380	488	544	601	978	858	978	699	779	766	764	740	978	1057
45.0	1111	294	431	558	619	692	1111	989	1111	889	990	965	959	951	1111	1359
50.0	1255	380	504	646	713	796	1255	1138	1255	1093	1215	1179	1180	1169	1255	1669
55.0	1400	478	631	750	827	925	1400	1322	1400	1323	1459	1426	1420	1403	1459	2004
60.0	1600	576	776	885	946	1075	1600	1536	1600	1574	1747	1704	1671	1669	1747	2385
65.0	1839	674	927	1061	1109	1224	1839	1749	1839	1839	2049	1998	1971	1962	2049	2803
70.0	2101	773	1079	1251	1317	1400	2101	2001	2101	2139	2366	2315	2294	2263	2366	3233
75.0	2383	871	1230	1456	1528	1633	2383	2332	2383	2455	2715	2649	2628	2583	2715	3690

For One Lane Carriageway

** Class A 1-Lane loading occupies 2.3 m width in a carriageway of 5.3 m and also includes an UDL of 500 Kg/m^2 on remaining width

Table 19 Shear Force for Simply Supported Span At 0.0 L Sec. for 1-Lane Carriageway: (for Steel Superstructure)

For Carriageway width not more than 5.3 m	Values are Inclusive with Impact and Lane Reduction Factor and a Over Load Factor of 1.4 for GVWs															
Span Length (m)	S.F. in (tonne) for Moving Traffic Condition								S.F. in (tonne) for Crowded / Traffic Jam Condition							
	Due to Governing IRC Load	Due to GVW 16.2T	Due to GVW 25.0T	Due to GVW 35.2T	Due to GVW 40.2T	Due to GVW 49.0T	Absolute Governing Shear For	Due to GVW 49 Ton With OLF=2	Due to Governing IRC Load	Due to GVW 16.2T	Due to GVW 25.0T	Due to GVW 35.2T	Due to GVW 40.2T	Due to GVW 49.0T	Absolute Governing Shear For	Due to GVW 49 Ton With OLF=2
10.0	64	26	36	39	39	36	64	52	64	26	30	31	32	32	64	45
15.0	82	27	39	46	49	48	82	69	82	35	41	39	40	42	82	60
20.0	93	27	40	50	54	58	93	82	93	43	49	49	50	50	93	71
25.0	98	29	40	52	57	62	98	89	98	52	59	58	59	59	98	85
30.0	100	32	44	53	58	65	100	92	100	61	69	68	68	68	100	98
35.0	101	35	49	57	61	66	101	94	101	69	78	77	77	78	101	111
40.0	102	37	53	63	66	71	102	101	102	78	88	86	87	87	102	124
45.0	103	38	55	68	72	75	103	108	103	87	98	95	96	96	103	137
50.0	109	42	58	72	78	83	109	118	109	95	107	105	105	105	109	150
55.0	119	45	62	76	82	89	119	128	119	104	117	114	114	114	119	163
60.0	128	48	67	80	86	95	128	136	128	113	127	124	123	123	128	176
65.0	136	50	71	85	90	100	136	143	136	121	136	133	133	133	136	189
70.0	142	53	75	91	96	104	142	149	142	130	146	142	142	142	146	203
75.0	148	56	78	96	102	110	148	157	148	139	155	152	151	151	155	216

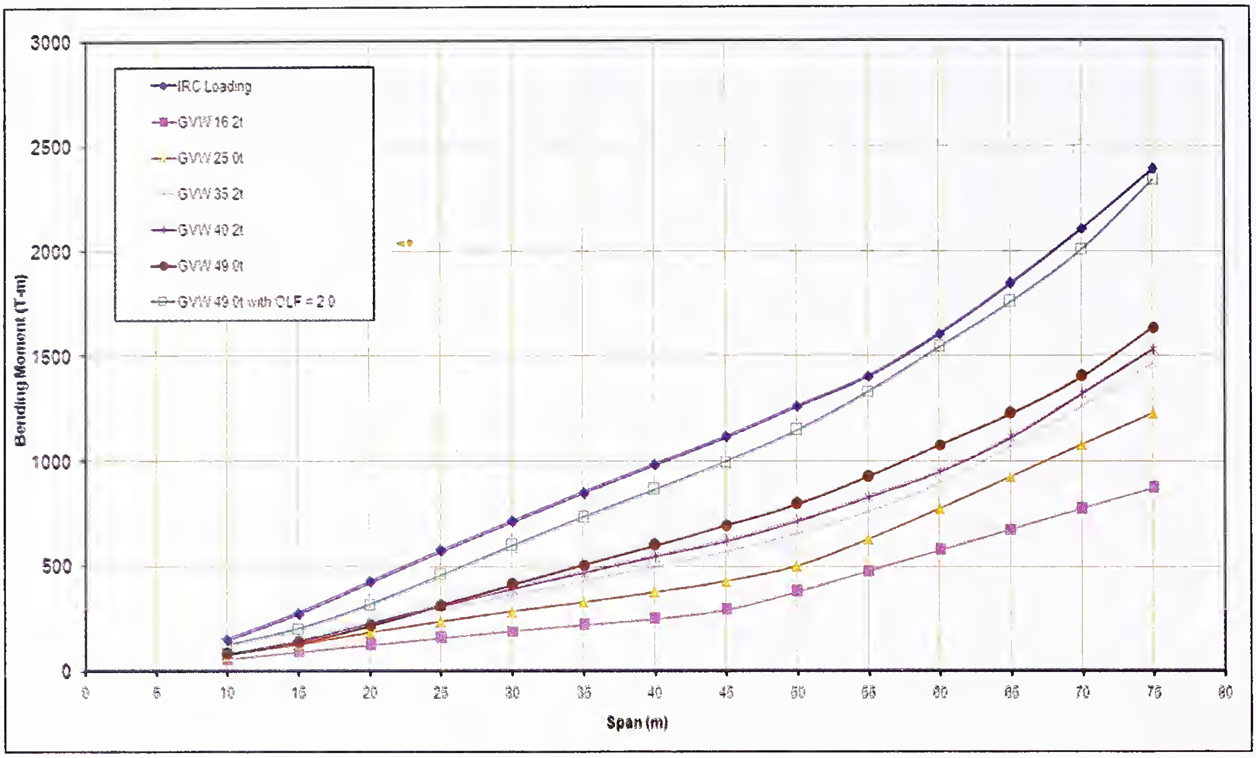


Fig. 29 Comparison of BM for One Lane with Steel Superstructure
(For Moving Traffic Condition)

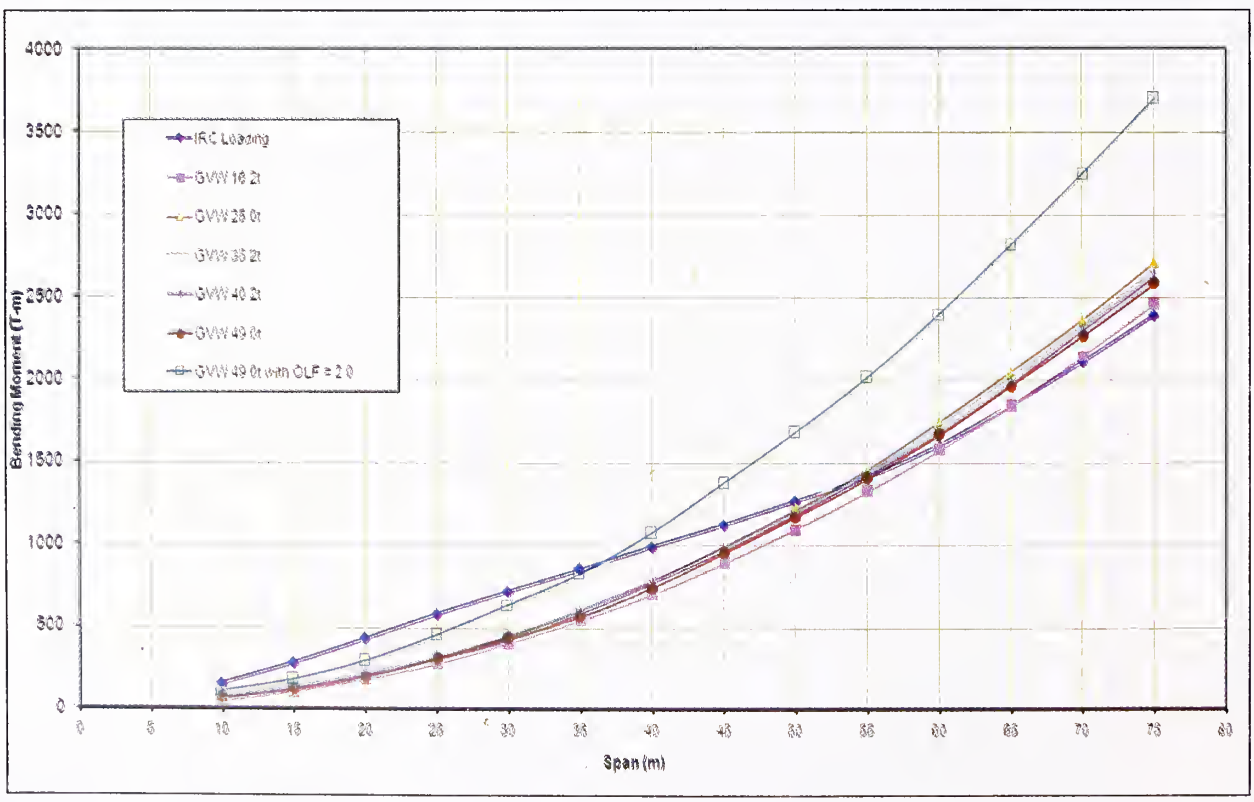


Fig. 30 Comparison of BM for One Lane with Steel Superstructure
(For Crowded/Traffic Jam Condition)

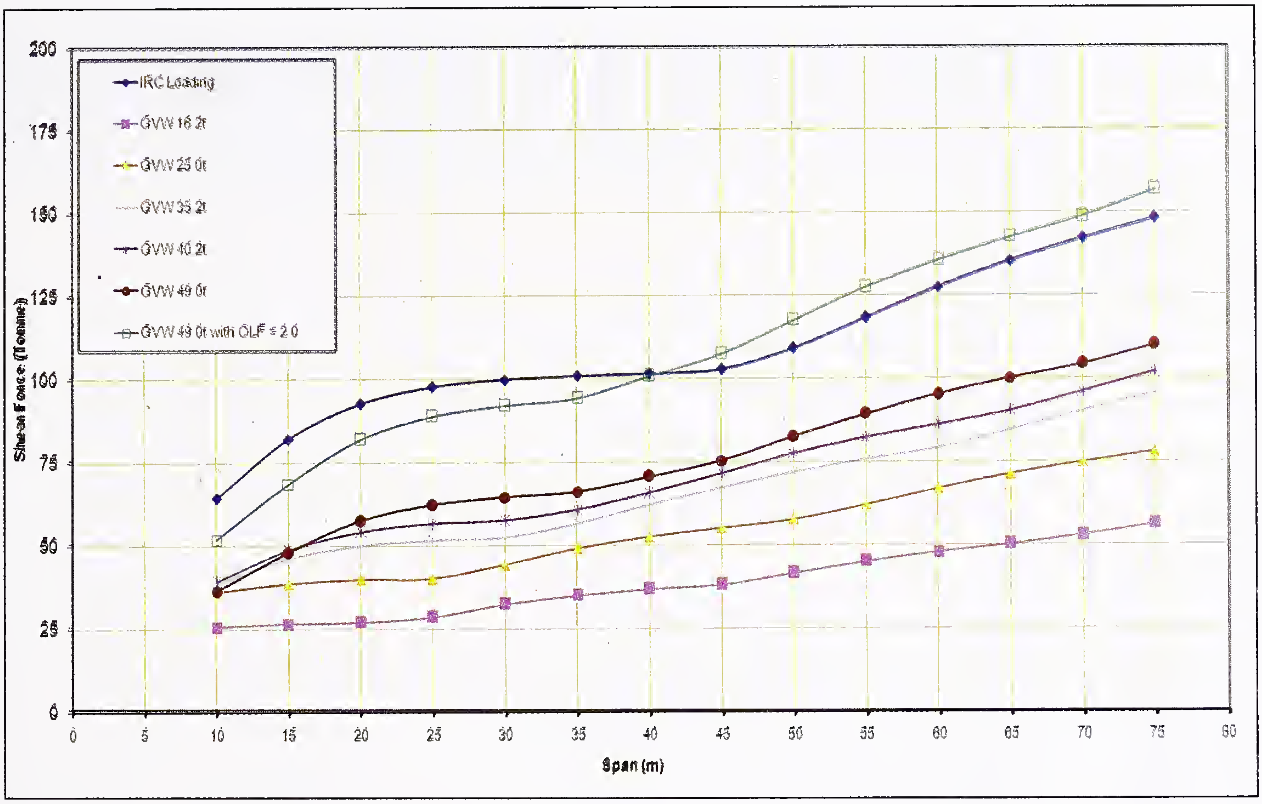


Fig. 31 Comparison of SF for One Lane with Steel Superstructure
(For Moving Traffic Condition)

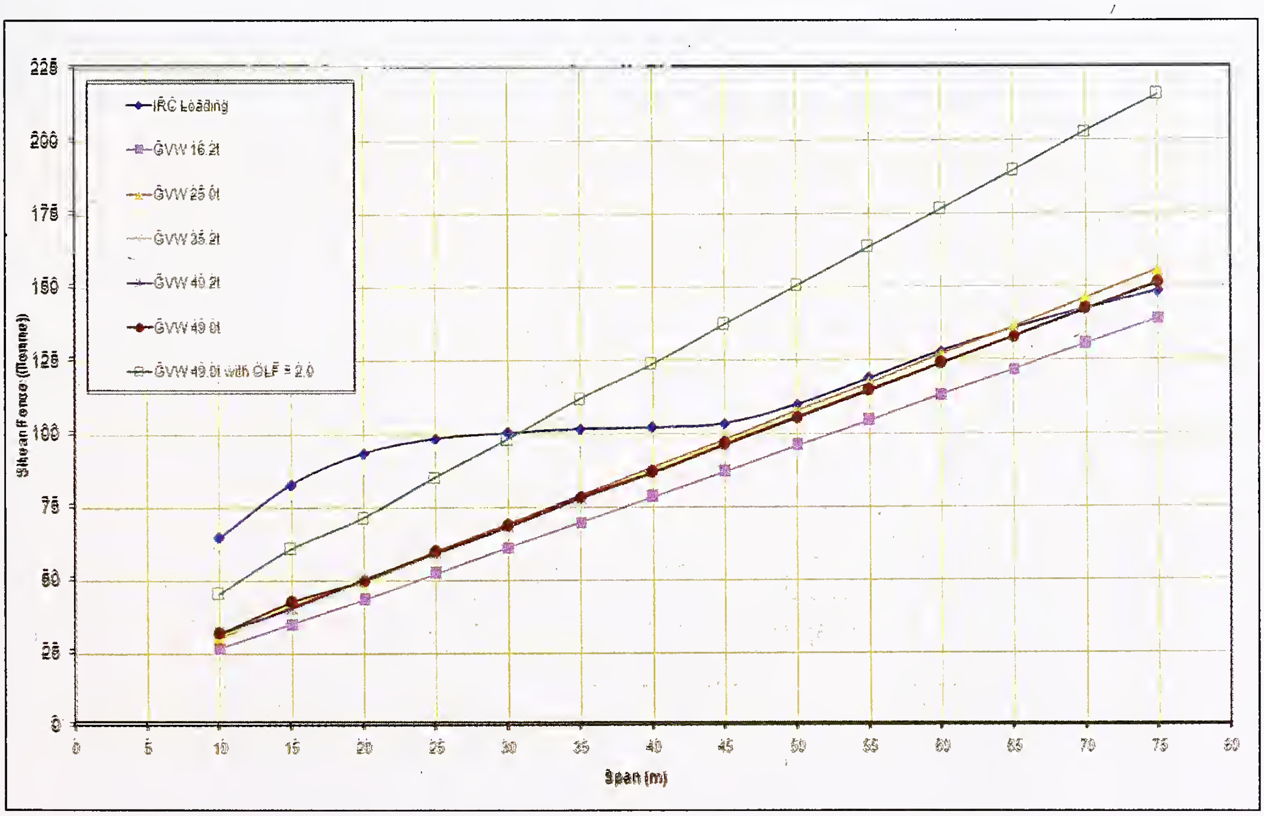


Fig. 32 Comparison of SF for One Lane with Steel Superstructure
(For Crowded/Traffic Jam Condition)

Table 20 Bending Moment for Simply Supported Span at 0.5 L Sec for 2-Lane Carriageway: (for Steel Superstructure)

For Carriageway width Between 5.3 m to 9.6 m Values are Inclusive with Impact and Lane Reduction Factor and a Over Load Factor of 1.4 for GVWs

Span Length (m)	B.M. in (t-m) for Moving Traffic Condition								B.M. in (t-m) for Crowded / Traffic Jam Condition							
	Due to Governing IRC Load	Due to GVW 16.2T	Due to GVW 25.0T	Due to GVW 35.2T	Due to GVW 40.2T	Due to GVW 49.0T	Absolute Governing Moment	Due to GVW 49 Ton With OLF=2	Due to Governing IRC Load	Due to GVW 16.2T	Due to GVW 25.0T	Due to GVW 35.2T	Due to GVW 40.2T	Due to GVW 49.0T	Absolute Governing Moment	Due to GVW 49 Ton With OLF=2
10.0	149	115	157	148	171	171	171	244	149	101	125	123	137	137	149	195
15.0	273	186	266	276	295	276	295	394	273	207	231	239	249	238	273	340
20.0	422	257	376	428	461	435	461	622	422	364	394	403	414	395	422	564
25.0	571	324	478	574	629	634	634	905	571	551	629	594	612	616	629	881
30.0	709	385	574	710	785	827	827	1181	709	798	894	863	837	869	894	1241
35.0	845	445	667	844	938	1016	1016	1451	845	1081	1187	1179	1163	1135	1187	1622
40.0	978	505	760	975	1088	1201	1201	1716	978	1398	1559	1532	1528	1480	1559	2114
45.0	1115	589	862	1116	1237	1384	1384	1978	1115	1778	1981	1929	1918	1903	1981	2718
50.0	1275	760	1008	1292	1426	1593	1593	2275	1275	2187	2430	2359	2361	2337	2430	3339
55.0	1451	956	1261	1500	1654	1851	1851	2644	1451	2646	2917	2852	2839	2806	2917	4008
60.0	1642	1153	1552	1770	1892	2150	2150	3071	1642	3147	3493	3408	3343	3339	3493	4769
65.0	1849	1349	1855	2122	2219	2449	2449	3498	1849	3678	4098	3997	3942	3924	4098	5606
70.0	2081	1545	2158	2503	2634	2801	2801	4001	2081	4279	4731	4629	4588	4526	4731	6466
75.0	2333	1741	2460	2912	3056	3265	3265	4664	2333	4909	5429	5299	5256	5166	5429	7380

Table 21 Shear Force for Simply Supported Span at 0.0 L Sec. for 2-Lane Carriageway: (for Steel Superstructure)

For Carriageway width Between 5.3 m to 9.6 m — Values are Inclusive with Impact and Lane Reduction Factor and a Over Load Factor of 1.4 for GVWS

Span Length (m)	S.F. in (tonne) for Moving Traffic Condition								S.F. in (tonne) for Crowded / Traffic Jam Condition							
	Due to Governing IRC Load	Due to GVW 16.2T	Due to GVW 25.0T	Due to GVW 35.2T	Due to GVW 40.2T	Due to GVW 49.0T	Absolute Governing Shear For	Due to GVW 49 Ton With OLF=2	Due to Governing IRC Load	Due to GVW 16.2T	Due to GVW 25.0T	Due to GVW 35.2T	Due to GVW 40.2T	Due to GVW 49.0T	Absolute Governing Shear For	Due to GVW 49 Ton With OLF=2
10.0	67	51	72	78	79	73	79	104	67	53	60	62	64	63	67	90
15.0	82	53	77	93	98	96	98	138	82	69	81	78	80	85	85	121
20.0	93	54	80	100	109	115	115	164	93	86	99	98	100	99	100	142
25.0	98	57	80	104	113	125	125	178	98	104	119	116	118	119	119	170
30.0	100	65	89	105	115	129	129	185	100	121	138	135	136	137	138	195
35.0	101	70	98	114	122	132	132	189	101	138	157	154	155	156	157	223
40.0	102	74	105	125	132	142	142	202	102	156	176	172	173	173	176	247
45.0	104	76	110	135	143	151	151	215	104	173	195	191	191	192	195	274
50.0	111	83	115	144	155	165	165	236	111	191	215	210	210	210	215	301
55.0	121	90	124	152	165	179	179	256	121	208	233	228	229	229	233	327
60.0	132	96	134	159	173	190	190	272	132	225	253	247	247	247	253	353
65.0	141	100	142	170	181	200	200	286	141	243	272	266	266	265	272	379
70.0	149	106	149	181	192	209	209	298	149	260	292	285	284	284	292	406
75.0	156	112	156	192	204	220	220	314	156	278	311	304	302	302	311	431

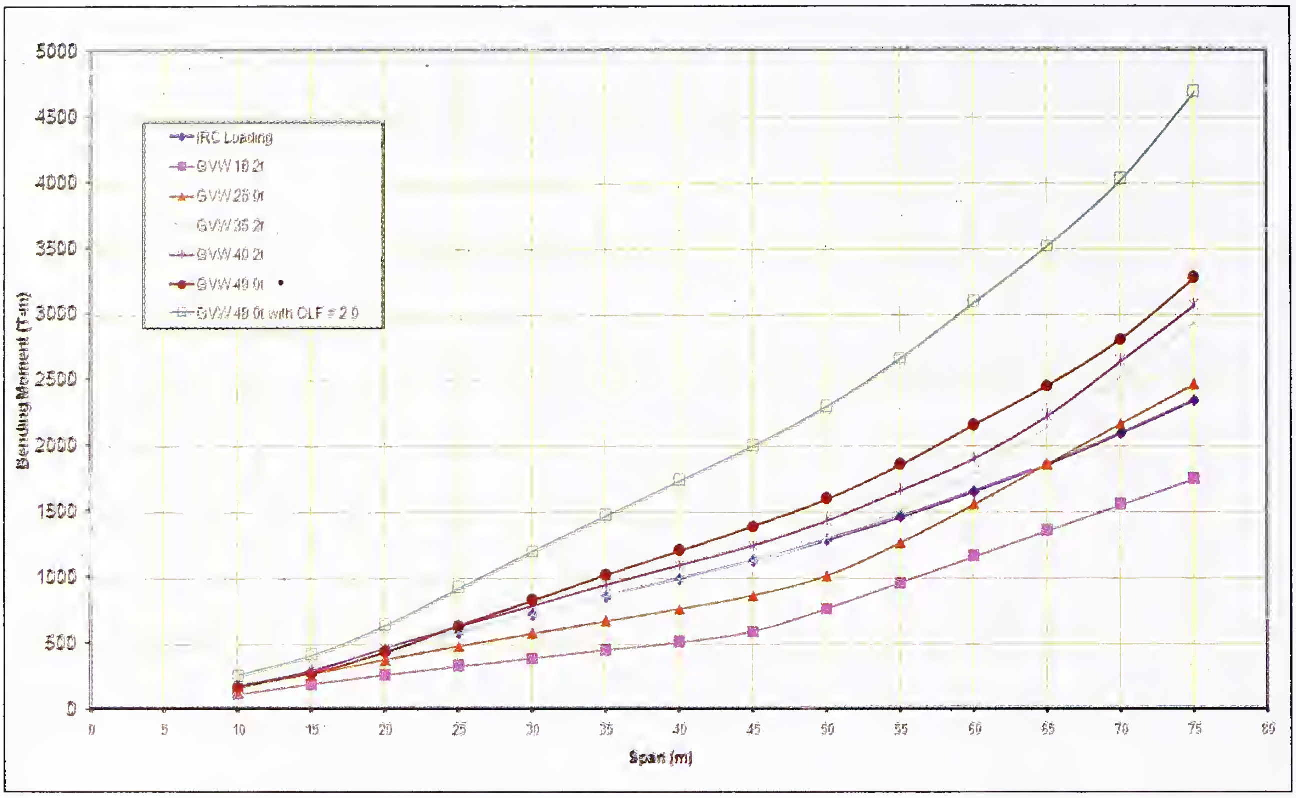


Fig. 33 Comparison of BM for Two Lanes with Steel Superstructure
(For Moving Traffic Condition)

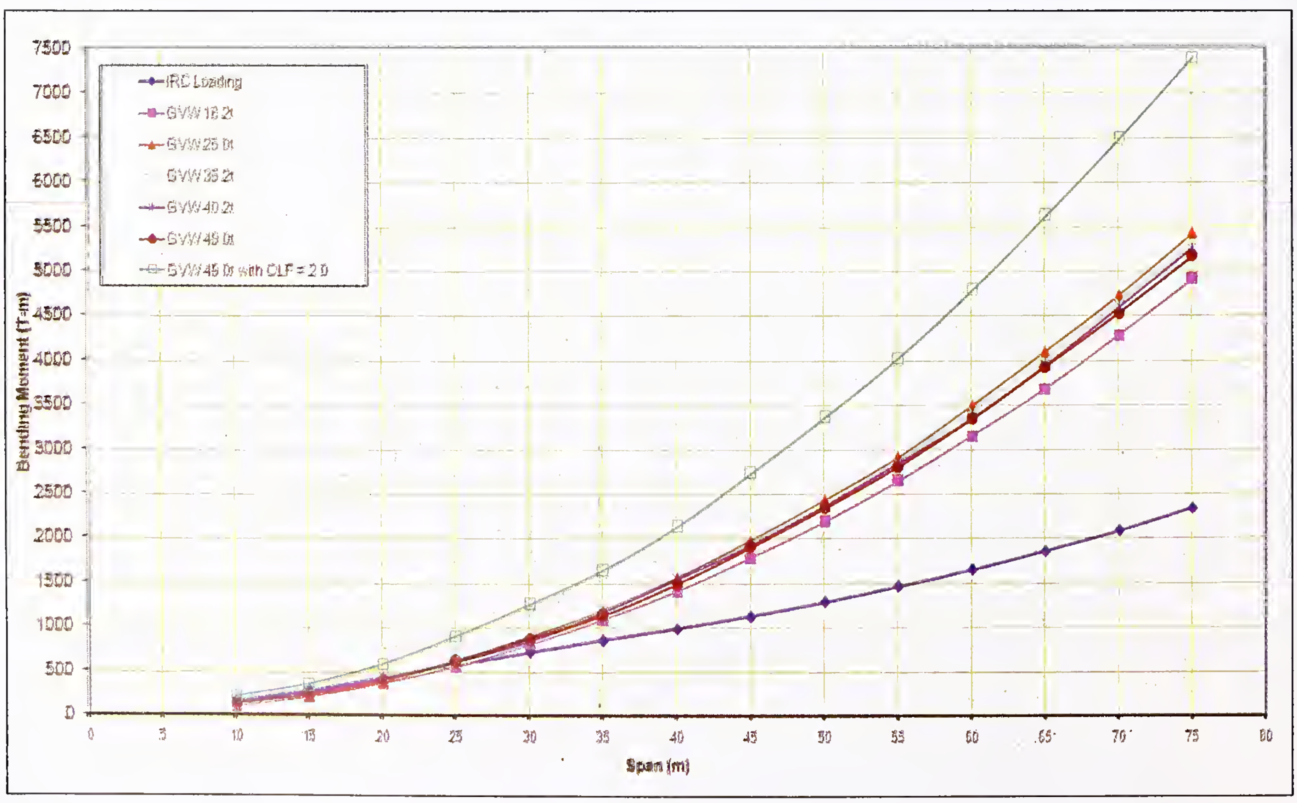


Fig. 34 Comparison of BM for Two Lanes with Steel Superstructure
(For Crowded/Traffic Jam Condition)

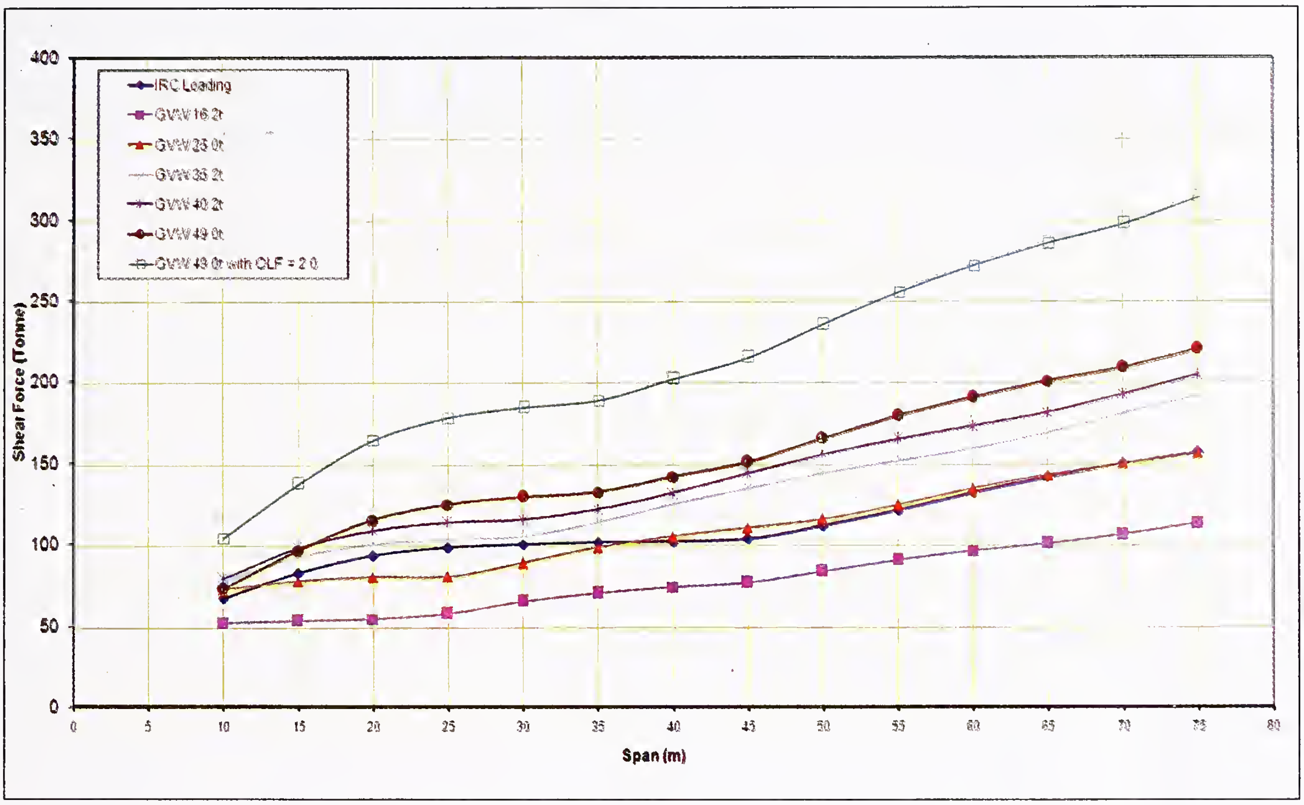


Fig. 35 Comparison of SF for Two Lanes with Steel Superstructure
(For Moving Traffic Condition)

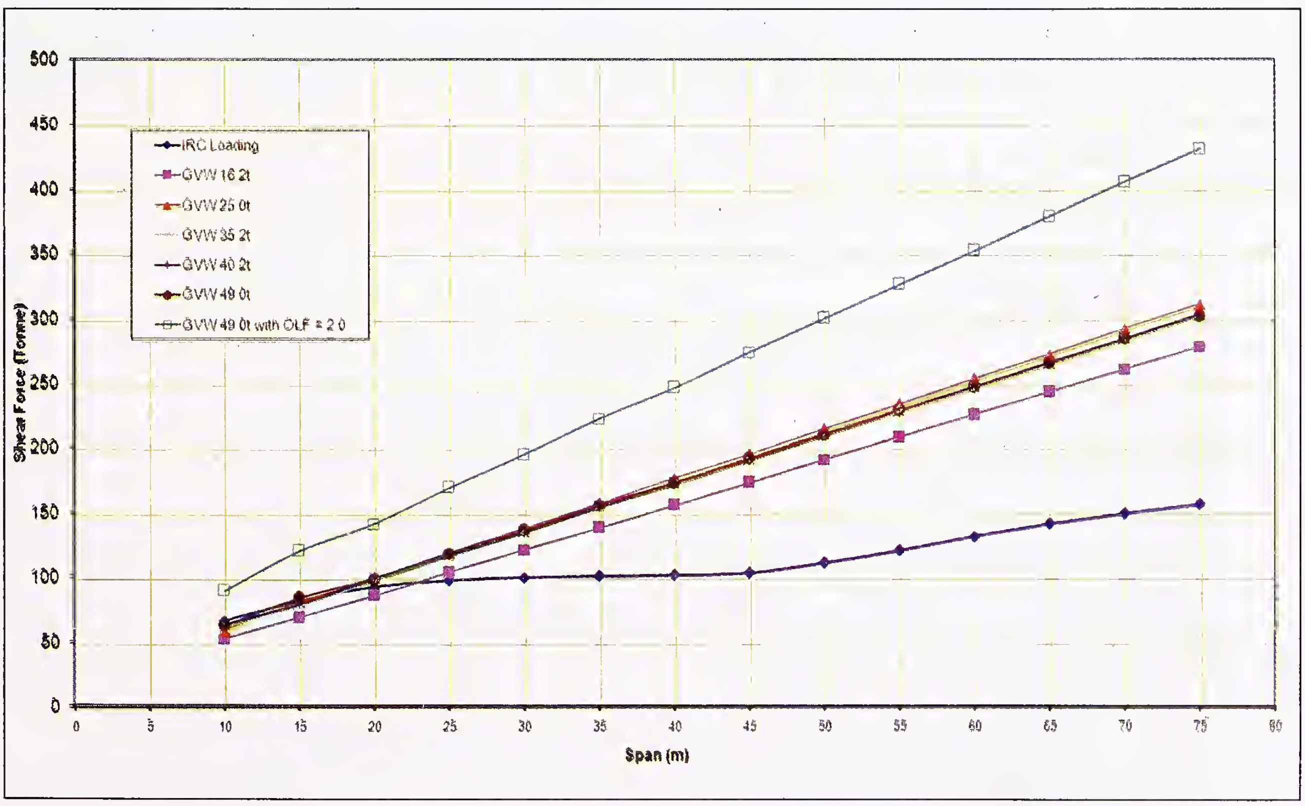


Fig. 36 Comparison of SF for Two Lanes with Steel Superstructure
(For Crowded/Traffic Jam Condition)

Table 22 Bending Moment for Simply Supported Span at 0.5 L Sec for 3-Lane Carriageway: (for Steel Superstructure)

For Carriageway width Between 9.6 m to 13.1 m   Values are Inclusive with Impact and Lane Reduction Factor and a Over Load Factor of 1.4 for GVWs

Span Length (m)	B.M. in (t-m) For Moving Traffic Condition								B.M. in (t-m) For Crowded / Traffic Jam Condition							
	Due to Governing IRC Load	Due to GVW 16.2T	Due to GVW 25.0T	Due to GVW 35.2T	Due to GVW 40.2T	Due to GVW 49.0T	Absolute Governing Moment	Due to GVW 49 Ton With OLF=2	Due to Governing IRC Load	Due to GVW 16.2T	Due to GVW 25.0T	Due to GVW 35.2T	Due to GVW 40.2T	Due to GVW 49.0T	Absolute Governing Moment	Due to GVW 49 Ton With OLF=2
10.0	200	156	212	200	231	231	231	329	200	136	169	166	184	184	200	264
15.0	361	251	359	373	398	372	398	532	361	279	311	322	336	322	361	460
20.0	551	347	507	577	623	588	623	840	551	491	532	545	559	533	559	762
25.0	746	437	646	775	849	856	856	1222	746	743	850	802	826	832	850	1189
30.0	938	520	774	959	1059	1116	1116	1595	938	1077	1207	1165	1130	1173	1207	1676
35.0	1128	601	901	1139	1266	1371	1371	1959	1128	1459	1603	1592	1570	1533	1603	2189
40.0	1316	682	1026	1317	1469	1622	1622	2317	1316	1888	2104	2069	2063	1998	2104	2854
45.0	1506	795	1164	1507	1670	1869	1869	2670	1506	2400	2674	2605	2589	2568	2674	3669
50.0	1722	1026	1360	1745	1925	2150	2150	3071	1722	2952	3281	3185	3187	3155	3281	4508
55.0	1959	1291	1702	2025	2233	2499	2499	3569	1959	3572	3938	3850	3833	3788	3938	5411
60.0	2217	1556	2095	2390	2554	2902	2902	4146	2217	4249	4716	4601	4513	4507	4716	6439
65.0	2496	1821	2504	2865	2995	3306	3306	4722	2496	4965	5532	5396	5321	5298	5532	7568
70.0	2809	2086	2913	3379	3556	3781	3781	5402	2809	5776	6387	6249	6194	6110	6387	8729
75.0	3149	2351	3322	3931	4125	4408	4408	6297	3149	6628	7329	7153	7095	6974	7329	9963

Table 23 Shear Force for Simply Supported Span at 0.0 L Sec. for 3-Lane Carriageway: (for Steel Superstructure)

For Carriageway width Between 9.6 m to 13.1 m — Values are Inclusive with Impact and Lane Reduction Factor and a Over Load Factor of 1.4 for GVWs

Span Length (m)	S.F. in (tonne) for Moving Traffic Condition								S.F. in (tonne) for Crowded/Traffic Jam Condition							
	Due to Governing IRC Load	Due to GVW 16.2T	Due to GVW 25.0T	Due to GVW 35.2T	Due to GVW 40.2T	Due to GVW 49.0T	Absolute Governing Shear For	Due to GVW 49 Ton With OLF=2	Due to Governing IRC Load	Due to GVW 16.2T	Due to GVW 25.0T	Due to GVW 35.2T	Due to GVW 40.2T	Due to GVW 49.0T	Absolute Governing Shear For	Due to GVW 49 Ton With OLF=2
10.0	90	69	97	105	106	98	106	140	90	71	81	84	86	85	90	122
15.0	104	72	104	125	132	130	132	186	104	93	110	106	108	114	114	163
20.0	117	73	108	136	147	155	155	222	117	116	134	132	135	134	135	191
25.0	126	77	108	140	153	168	168	240	126	140	161	157	159	160	161	229
30.0	131	88	120	142	155	174	174	249	131	164	186	182	183	184	186	264
35.0	134	95	133	154	164	179	179	255	134	187	212	208	209	211	212	301
40.0	136	99	142	169	178	191	191	273	136	210	238	232	234	234	238	334
45.0	140	103	149	182	194	204	204	291	140	234	263	258	258	259	263	370
50.0	150	113	156	195	209	223	223	319	150	257	290	283	284	284	290	406
55.0	163	122	168	205	222	242	242	345	163	281	315	308	309	309	315	441
60.0	178	129	181	215	233	257	257	367	178	304	342	334	333	333	342	476
65.0	191	136	192	229	244	270	270	386	191	327	367	359	358	358	367	512
70.0	202	143	202	245	259	282	282	402	202	351	394	384	384	383	394	548
75.0	211	152	210	259	275	297	297	424	211	375	419	410	408	407	419	582

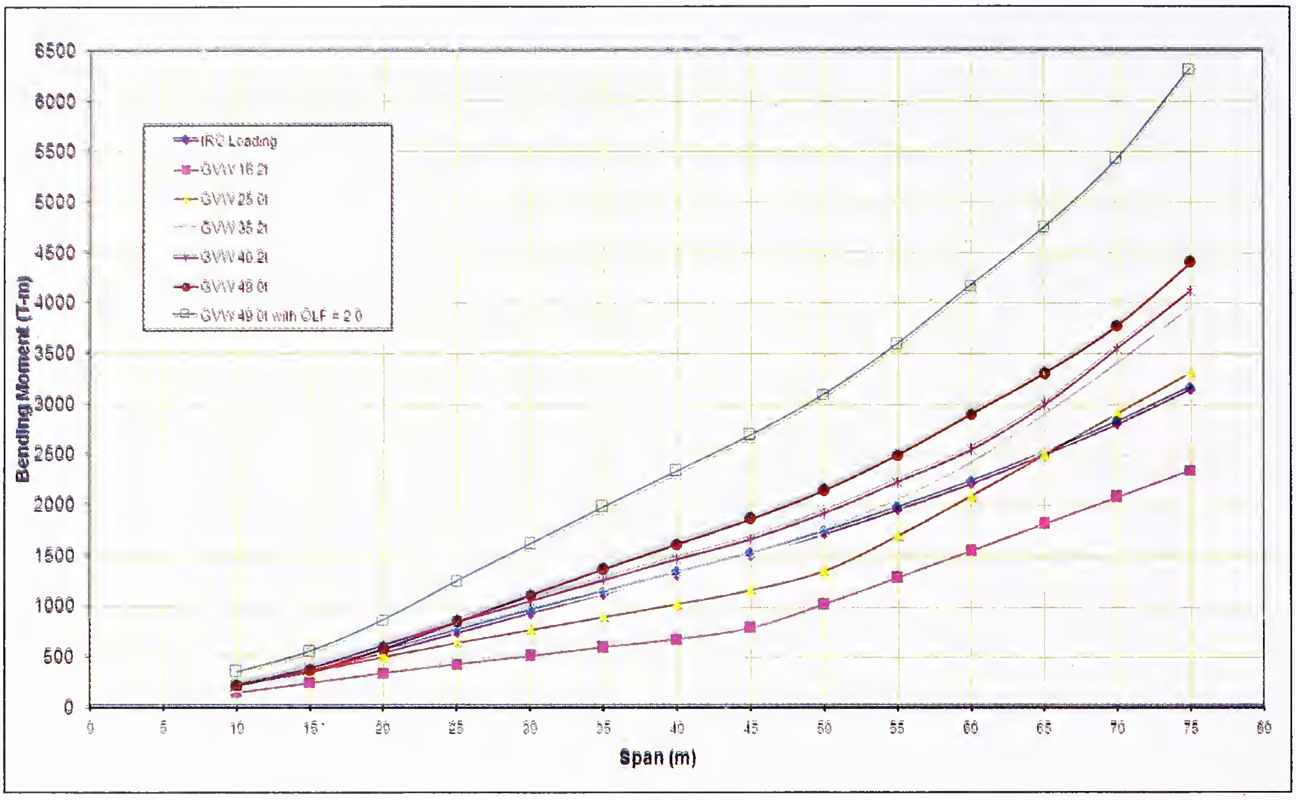


Fig. 37 Comparison of BM for Three Lanes with Steel Superstructure (For Moving Traffic Condition)

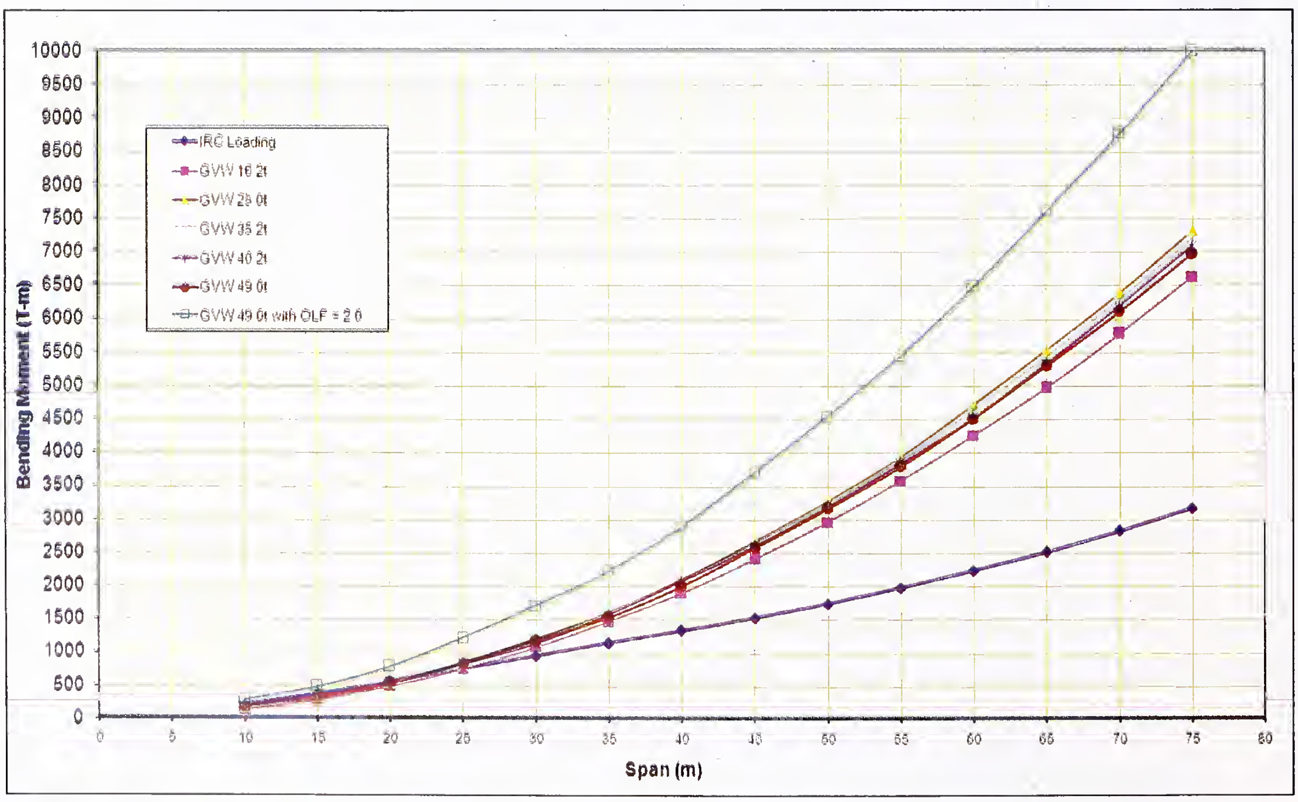


Fig. 38 Comparison of BM for Three Lanes with Steel Superstructure (For Crowded/Traffic Jam Condition)

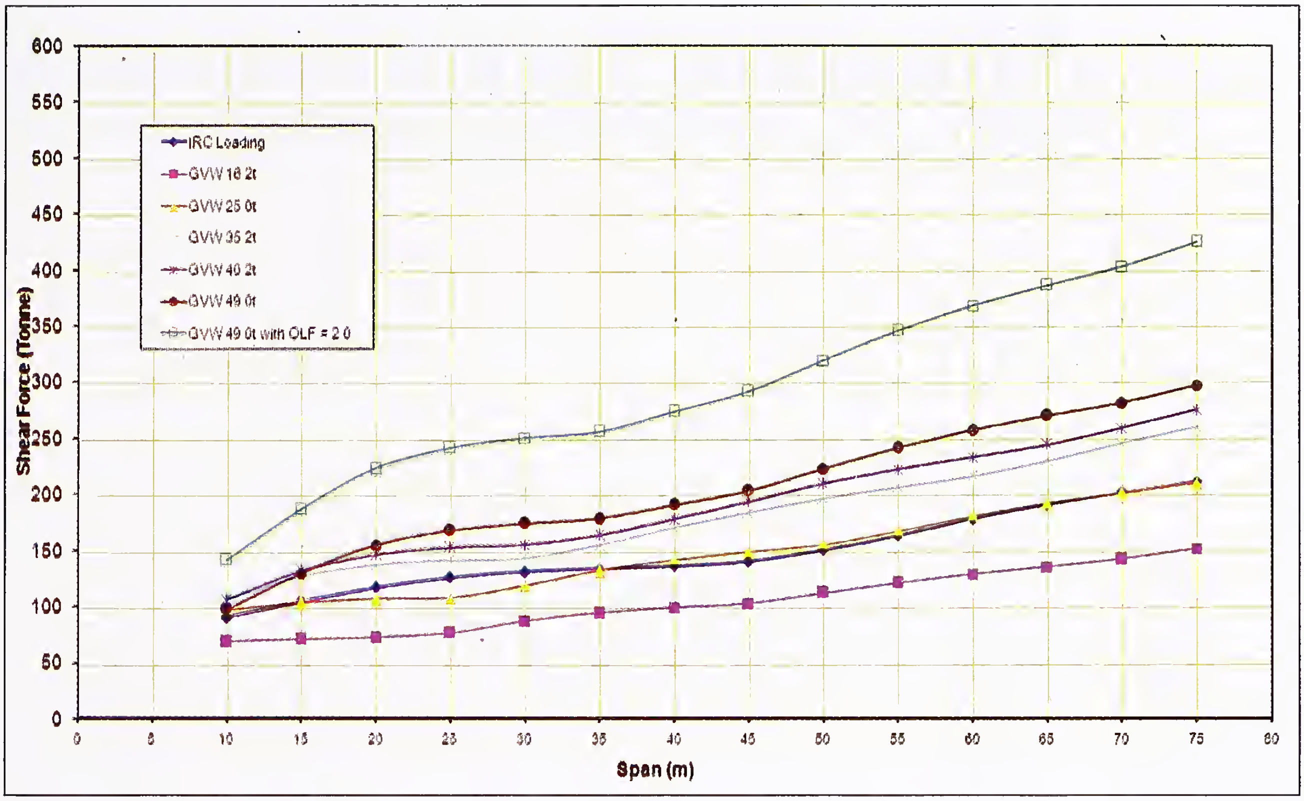


Fig. 39 Comparison of SF for Three Lanes with Steel Superstructure
(For Moving Traffic Condition)

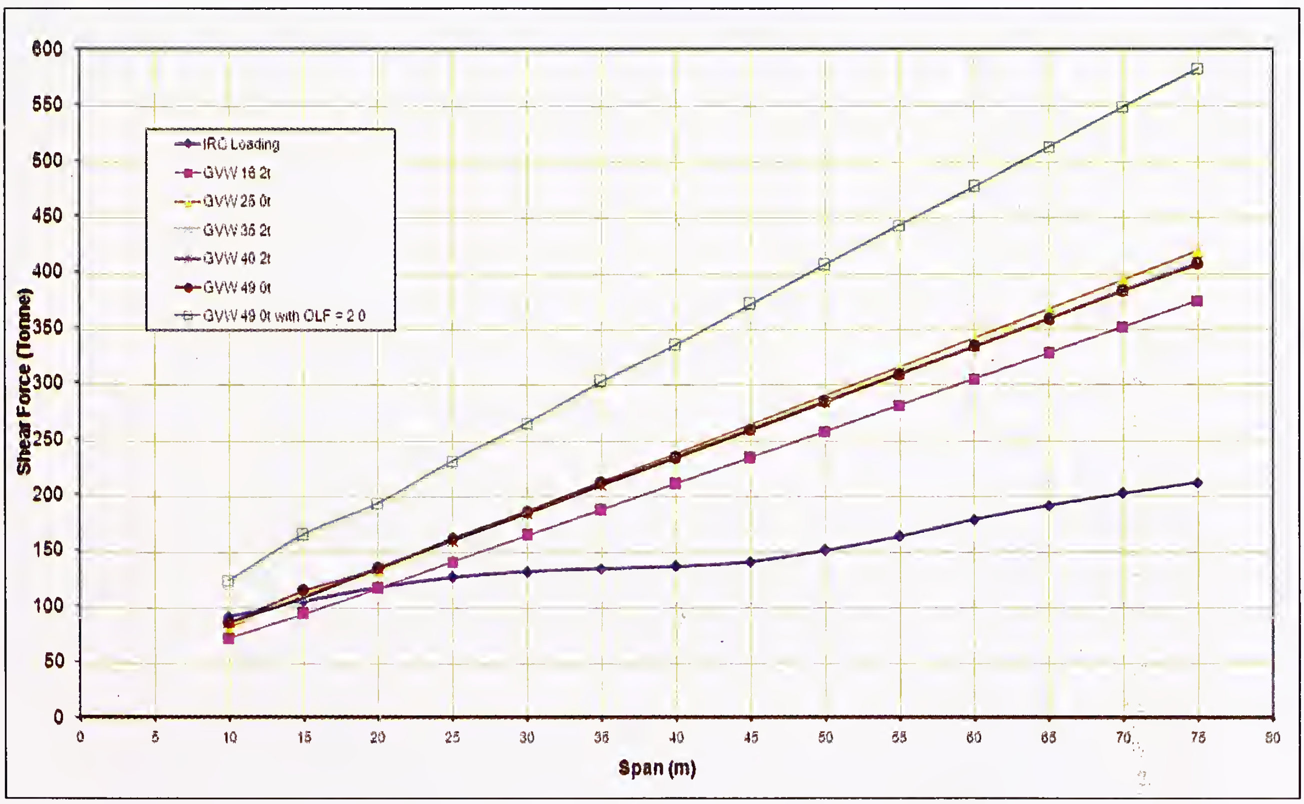


Fig. 40 Comparison of SF for Three Lanes with Steel Superstructure
(For Crowded/Traffic Jam Condition)

Table 24 Bending Moment for Simply Supported Span at 0.5 L Sec for 4-Lane Carriageway: (for Steel Superstructure)

For Carriageway width Between 13.1 m to 19.6 m   Values are Inclusive with Impact and Lane Reduction Factor and a Over Load Factor of 1.4 for GVWs

Span Length (m)	B.M. in (t-m) for Moving Traffic Condition								B.M. in (t-m) for Crowded / Traffic Jam Condition							
	Due to Governing IRC Load	Due to GVW 16.2T	Due to GVW 25.0T	Due to GVW 35.2T	Due to GVW 40.2T	Due to GVW 49.0T	Absolute Governing Moment	Due to GVW 49 Ton With OLF=2	Due to Governing IRC Load	Due to GVW 16.2T	Due to GVW 25.0T	Due to GVW 35.2T	Due to GVW 40.2T	Due to GVW 49.0T	Absolute Governing Moment	Due to GVW 49 Ton With OLF=2
10.0	238	185	251	237	273	273	273	390	238	161	201	196	219	219	238	312
15.0	437	298	426	442	471	441	471	630	437	330	369	382	398	381	437	545
20.0	676	411	601	684	738	697	738	995	676	582	631	645	663	632	676	903
25.0	913	518	765	918	1006	1014	1014	1449	913	881	1007	951	978	986	1007	1409
30.0	1135	616	918	1136	1256	1323	1323	1890	1135	1277	1430	1381	1339	1390	1430	1986
35.0	1352	713	1068	1350	1500	1625	1625	2322	1352	1729	1900	1887	1860	1816	1900	2595
40.0	1566	808	1215	1560	1741	1922	1922	2746	1566	2237	2494	2452	2445	2368	2494	3383
45.0	1785	942	1379	1786	1980	2215	2215	3164	1785	2845	3169	3087	3069	3044	3169	4349
50.0	2041	1216	1612	2068	2281	2548	2548	3640	2041	3499	3888	3774	3777	3740	3888	5342
55.0	2322	1530	2018	2400	2646	2961	2961	4230	2322	4234	4668	4563	4543	4489	4668	6413
60.0	2627	1844	2483	2833	3026	3440	3440	4914	2627	5036	5589	5453	5348	5342	5589	7631
65.0	2959	2158	2967	3396	3550	3918	3918	5597	2959	5885	6557	6395	6307	6279	6557	8969
70.0	3329	2472	3452	4005	4214	4482	4482	6402	3329	6846	7570	7407	7342	7242	7570	10345
75.0	3732	2786	3937	4659	4889	5224	5224	7463	3732	7855	8686	8478	8409	8266	8686	11808

Table 25 Shear Force for Simply Supported Span at 0.0 L Sec. for 4-Lane Carriageway: (for Steel Superstructure)

For Carriageway width Between 13.1 m to 19.6 m   Values are Inclusive with Impact and Lane Reduction Factor and a Over Load Factor of 1.4 for GVWs

Span Length (m)	S.F. in (tonne) For Moving Traffic Condition								S.F. in (tonne) For Crowded / Traffic Jam Condition							
	Due to Governing IRC Load	Due to GVW 16.2T	Due to GVW 25.0T	Due to GVW 35.2T	Due to GVW 40.2T	Due to GVW 49.0T	Absolute Governing Shear For	Due to GVW 49 With OLF=2 on	Due to Governing IRC Load	Due to GVW 16.2T	Due to GVW 25.0T	Due to GVW 35.2T	Due to GVW 40.2T	Due to GVW 49.0T	Absolute Governing Shear For	Due to GVW 49 Ton With OLF=2
10.0	107	82	115	124	126	116	126	166	107	84	96	99	102	101	107	144
15.0	132	85	124	148	157	154	157	220	132	111	130	125	128	135	135	193
20.0	149	86	128	161	174	184	184	263	149	138	158	157	159	159	159	227
25.0	157	92	128	166	181	199	199	285	157	166	190	186	188	190	190	272
30.0	160	104	142	168	184	207	207	295	160	194	220	216	217	219	220	312
35.0	162	112	157	182	194	212	212	302	162	221	251	246	247	250	251	357
40.0	163	118	168	200	211	227	227	324	163	249	282	275	277	277	282	396
45.0	166	122	176	216	229	241	241	345	166	277	312	306	306	307	312	439
50.0	178	133	185	231	248	264	264	378	178	305	344	336	336	337	344	481
55.0	193	144	199	243	263	286	286	409	193	333	374	365	366	366	374	523
60.0	211	153	214	255	276	305	305	435	211	360	405	396	395	395	405	564
65.0	226	161	228	271	289	320	320	457	226	388	435	425	425	424	435	606
70.0	239	169	239	290	307	334	334	477	239	416	467	456	455	454	467	649
75.0	250	180	249	307	326	352	352	503	250	444	497	486	484	483	497	690

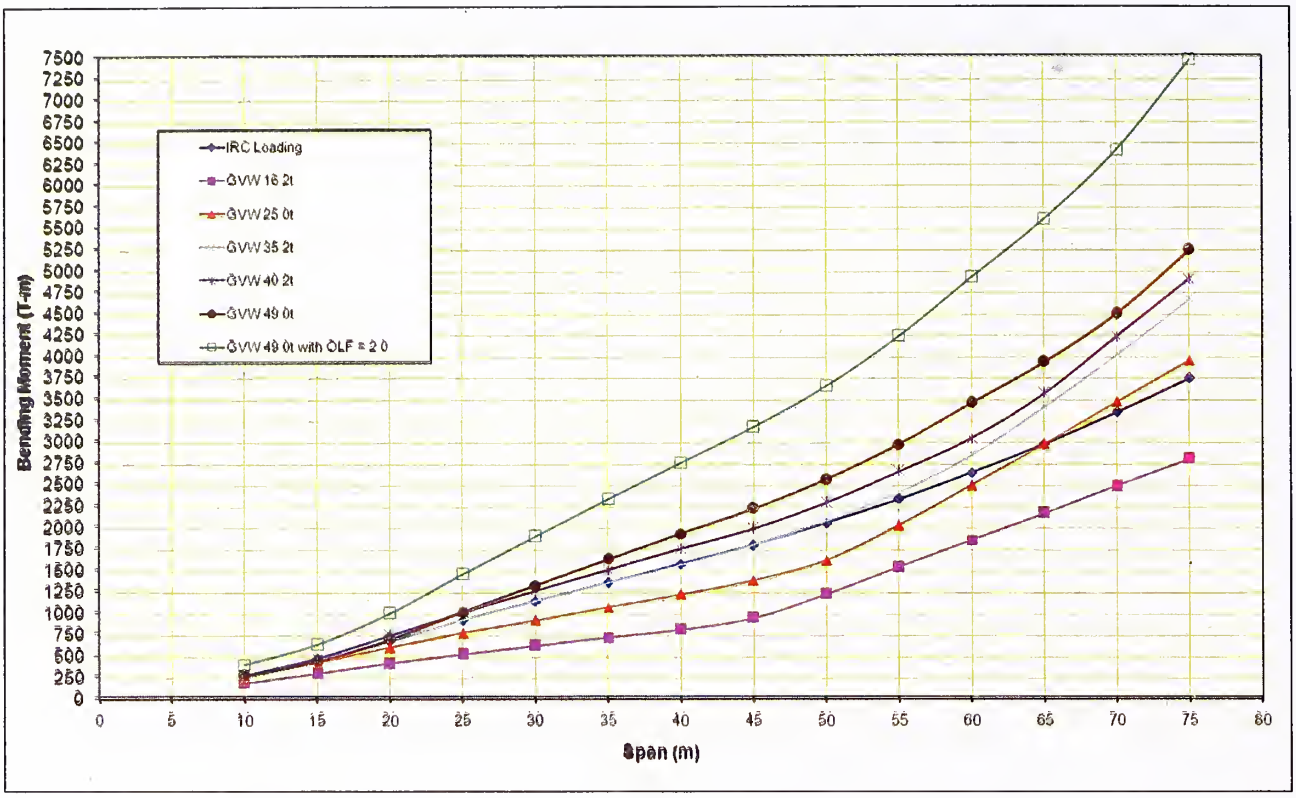


Fig. 41 Comparison of BM for Four Lanes with Steel Superstructure (For Moving Traffic Condition)

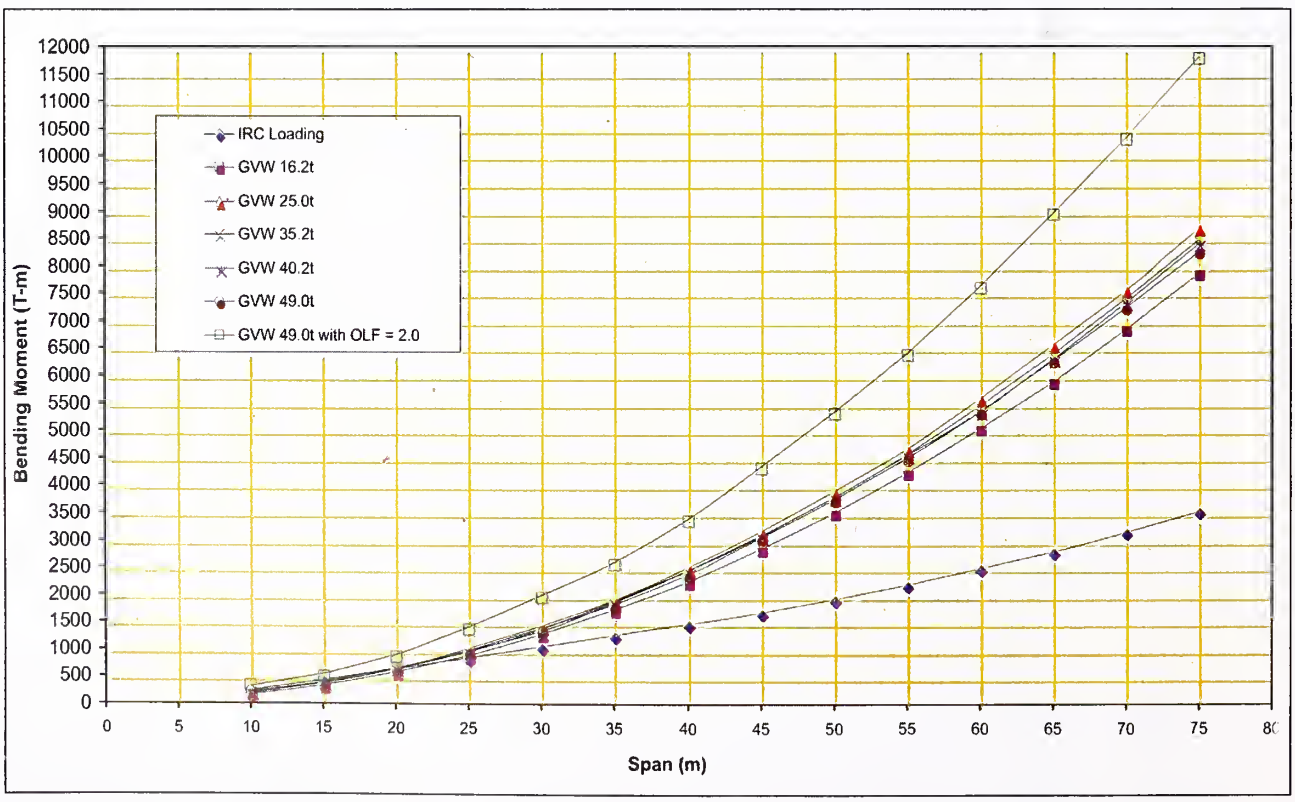


Fig. 42 Comparison of BM for IRC Four Lanes (For Crowded/Traffic Jam Condition)

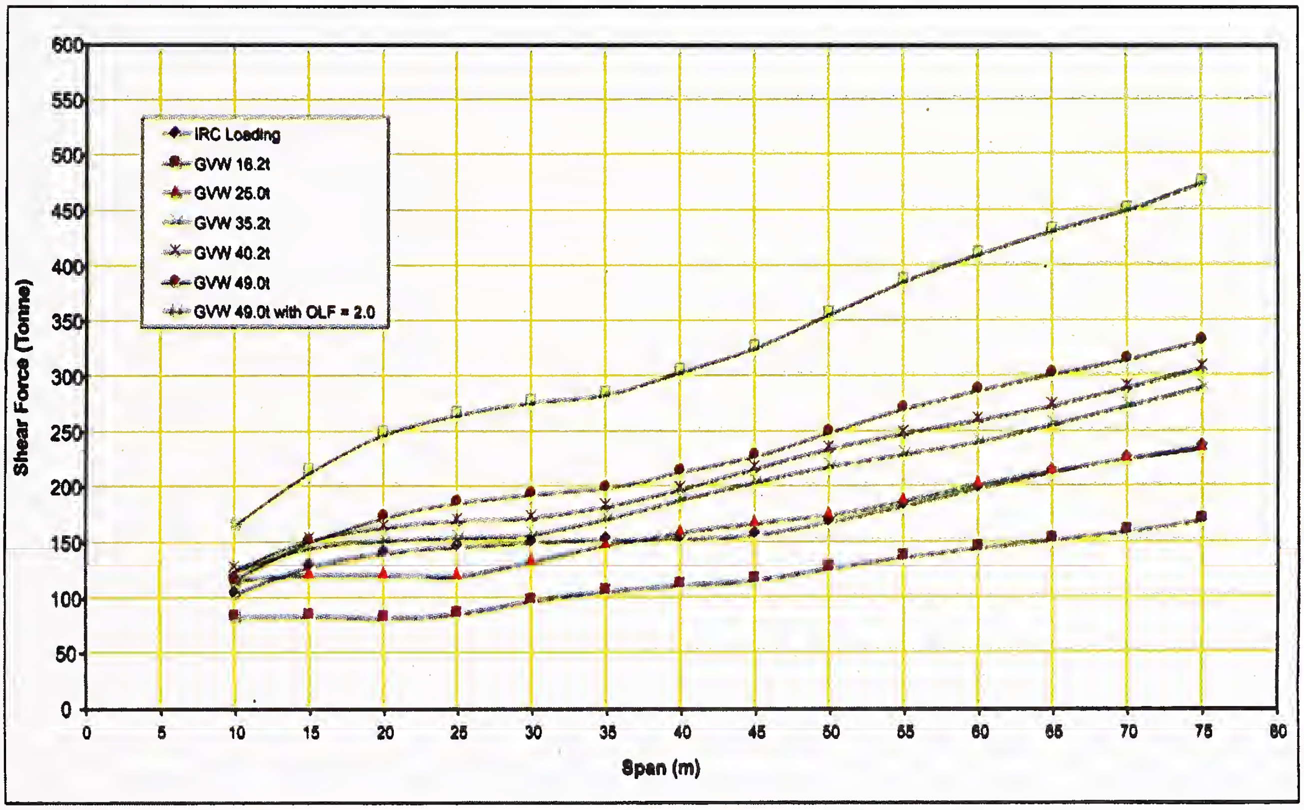


Fig. 43 Comparison of SF for Four Lanes with Steel Superstructure
(For Moving Traffic Condition)

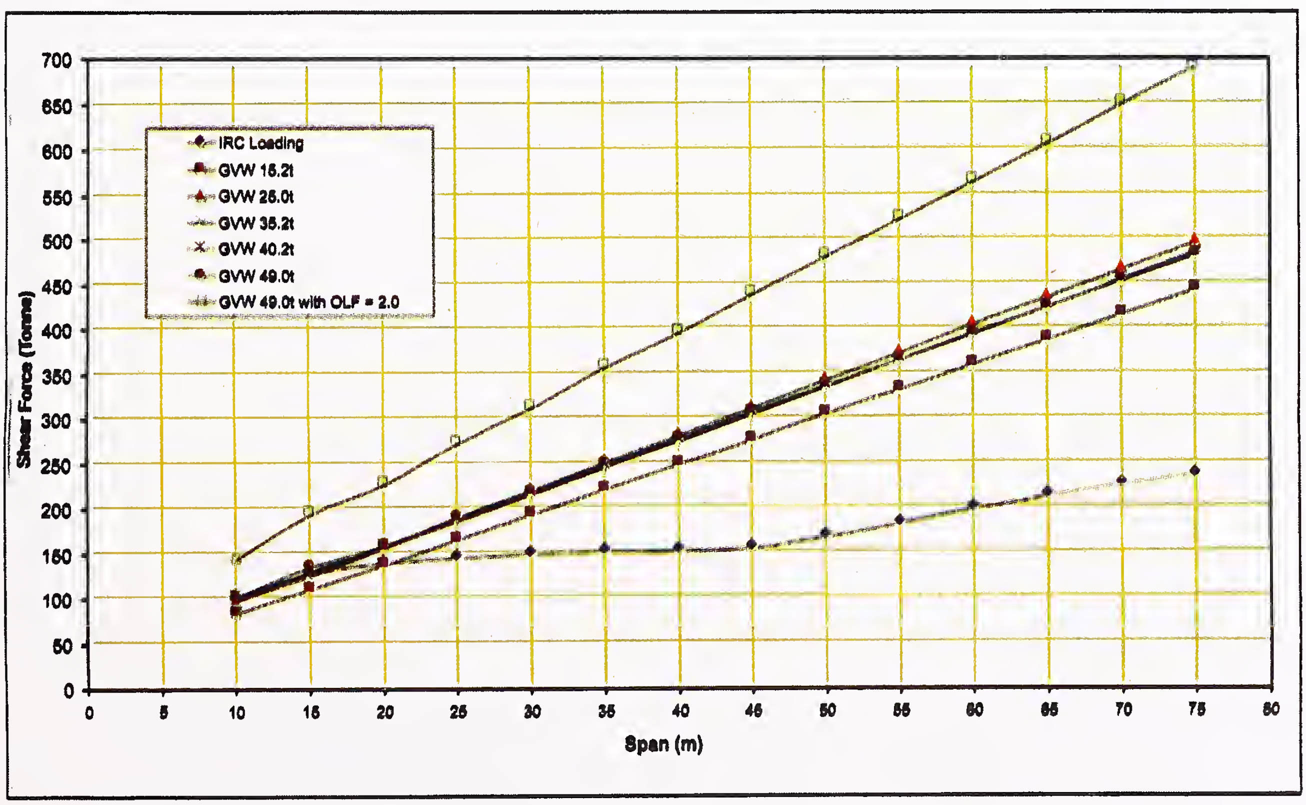


Fig. 44 Comparison of SF for Four Lanes with Steel Superstructure
(For Crowded/Traffic Jam Condition)

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(The Official amendments to this document would be published by the IRC in its periodical, 'Indian Highways which shall be considered as effective and as part of the code/guidelines/manual, etc. from the date specified therein)